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V. 56

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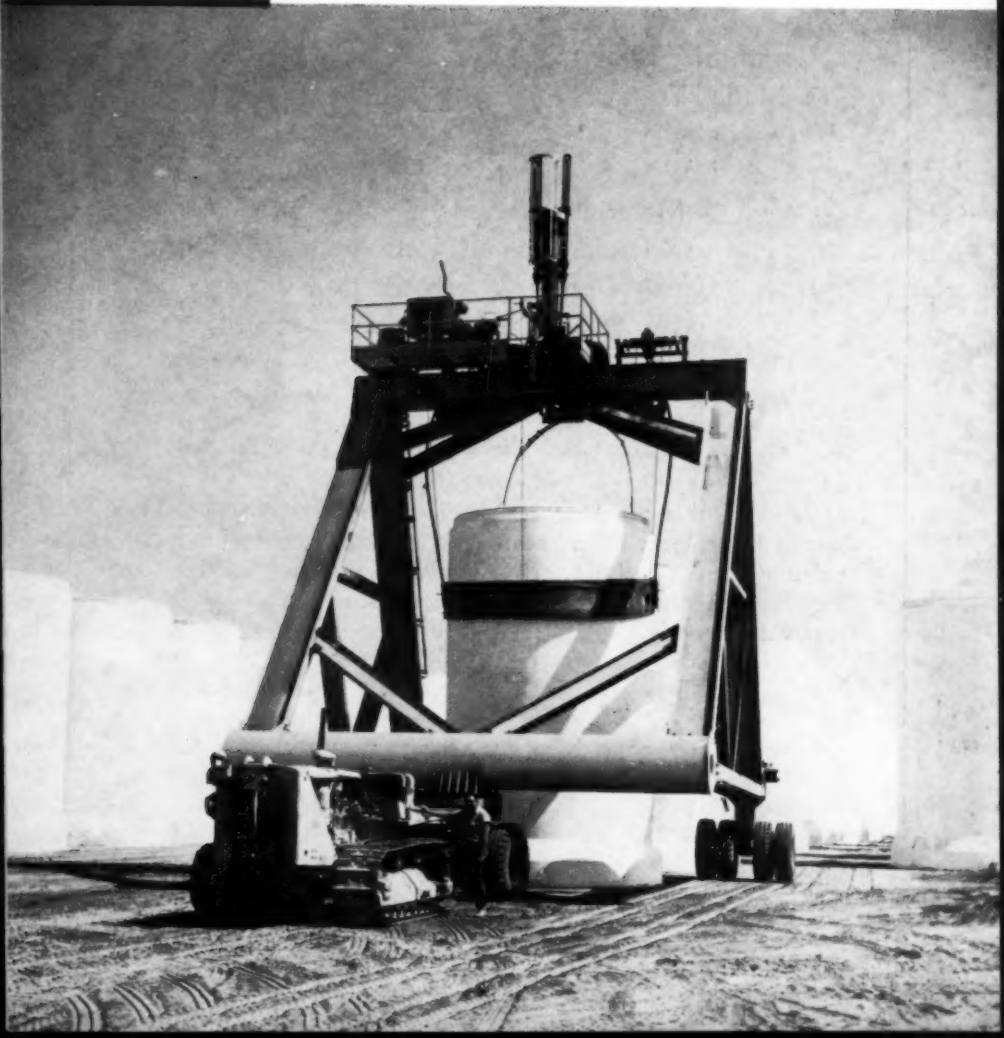

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I. L. TYLER opens the March issue with Chapter 12 of the Long-Time Study of Cement Performance in Concrete—"Concrete Exposed to Sea Water and Fresh Water."

Coauthors CLARENCE W. DUNHAM and HANS GESUND present a numerical method developed to determine the "Yield Moments of Reinforced Concrete Beams and Columns."

M. F. KAPLAN presents data from an experimental investigation conducted to determine the "Effects of Incomplete Compaction on the Compressive and Flexural Strength, Ultrasonic Pulse Velocity, and Dynamic Modulus of Elasticity of Concrete."

TUNG AU and DONALD L. BAIRD describe tests carried out to determine the "Bearing Capacity of Concrete Blocks."

Discussion of papers published in the July, August, and September, 1959, issues will appear in the March JOURNAL, in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

On the Cover—This 90-ton pipe handler was designed to handle 85 ton, 144 in. inside diameter, 24 ft sections of concrete pressure pipe. The pipe will be laid on the ocean bottom as part of the $20 million effluent outfall under construction for the Los Angeles Hyperion treatment plant.

The handler is 50 ft high, 32 ft wide, 35 ft long, and weighs 30 tons. The operator's platform is 35 ft above the ground. Various controls permit turning the pipe to a horizontal or vertical position. The handler-tractor combination lifts completed pipe from the base rings, transports it to the storage area, and from there to an ocean launching platform.

United Concrete Pipe Corp., Baldwin Park, Calif., has the subcontract for the pipe. General contractor on the project is Hyperion Constructors. — Photo courtesy American Concrete Pressure Pipe Association.

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Title No. 56-40

Design of Prestressed Lift Slabs for Deflection Control

By EDWARD K. RICE and FELIX KULKA

Prestressed concrete lift slabs have been used successfully, not only as an economical method of construction, but also as a medium to obtain flat slab buildings that have no deflection or camber.

The analysis of a flat slab can be extremely complex. The "beam method" has been developed as an approximate but applicable method of design. By this analysis, slabs may be designed for proper behavior and strength without the use of the more complicated plate theory.

■ LIFT-SLAB CONSTRUCTION was developed to save costs of forming and shoring (Fig. 1 and 2). Prestressing of flat slabs enables the slabs to carry heavier loads with shallower depths and fewer cracks. Neither the lift-slab method of construction nor the prestressed concrete slab is entirely satisfactory in itself: conventionally reinforced lift slabs show excellent economy, but in spans over 18 or 20 ft, in the absence of column capitals and drop panels, the slabs often unduly crack and deflect. On the other hand, cast-in-place prestressed slabs are crack-free and show excellent deflection control, but shortening and movement due to prestressing introduces complications in the details of the supports. When the two concepts of lift-slab and prestressed concrete are joined, however, their good qualities complement each other and their obvious faults cancel out.

BASIS OF DESIGN

In structural analysis and design it is always helpful to have a code at hand which gives convenient coefficients, so that the designer can use them in equations and come out with ready-made answers. When the analysis of the first prestressed lift slab was started, no conventional design procedure was available. By tracing reinforced concrete slab theory back to basic principles, using the laws of statics, it was found

Fig. 1—Slabs for six-story addition to the California Division of Highways Building, Los Angeles, shown stacked, during placing

that a prestressed concrete slab could be safely analyzed by the elastic theory, the allowable stresses being adjusted on the basis of its ultimate strength. Thus, the beam method was developed following the same concept used in reinforced concrete design. This method assumes that the slab acts as beams which span between column lines first in one direction, then in the other. Generally, full load is assigned to these beams, which then must be analyzed as statically indeterminate structures.

As far as the actual design of the prestressing is concerned, it, too, had to be tailored to a new condition. This is because the prestressing force in a continuous slab falls into an irregular pattern and its effective location is altered by linear transformation, which affects the results some 10-50 percent, and hence must be carefully considered.

These first assumptions, although based on observations and deductions as well as preliminary field tests, have since been substantiated not only by results of experiments in the laboratory, but also by the record of actual construction. The first laboratory test was a square slab supported on four corners simulating a simply supported lift slab.[1] The experimental results of stresses and deflections agreed well with those computed on the basis of the classic elastic theory. The ultimate strength was closely predicted by the yield-line theory. The latest piece of research in this field,[2] conducted by the University of California and the California State Division of Architecture, actually deals with a continuous slab. Again, close agreement is found between experimental and

ACI member **EDWARD K. RICE** is a partner in the consulting firm of T. Y. Lin and Associates, Los Angeles. Prior to this he was partner and chief engineer of Western Concrete Structures Co. for whom he developed many new techniques in prestressed lift-slab construction. He was a lecturer in engineering at the University of California for several years.

FELIX KULKA, ACI member, joined T. Y. Lin and Associates in 1955; he is now an associate with that firm. Active in the field of structural engineering for the past 10 years, his work is now focused to the design and the development of prestressed structures, particularly slab shells.

calculated values, particularly with respect to deflections and ultimate strength.

In the way of field testing, several completed structures have been test-loaded in excess of design loading, and it is found that calculated deflections, based on the elastic theory, agree closely with those measured in the field. Also, field observations have greatly aided in gaining proper insight into the general behavior of the slab.[3,4]

A large measure of success has been achieved in controlling deflections and limiting them to minimum values. It must be pointed out, however, that these observations are limited to solid slabs, and their conclusions may or may not apply to slabs of the waffle or joist type, which are much more sensitive to unusual loadings and stress distributions. Furthermore, for spans of varying proportions and with irregular column spacings or hole locations, great care must be exercised. This does not mean that such slabs cannot be designed, except that their design must be based on proper judgment and experience, rather than the blind adherence to empirical rules.

With the research data now available and with the observation of some 40 structures in California alone, rules can be established for the design of prestressed concrete lift slabs of the solid type and of regular patterns and proportions. No longer can they be designed for conventional loads and allowable stresses, as is usually done for reinforced concrete slabs. Analysis must be based on two criteria: strength, and proper behavior, which will be discussed separately.

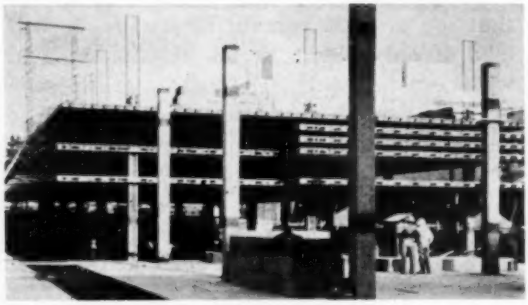


Fig. 2—Slabs for California Division of Highways Building being lifted into position

DESIGN FOR STRENGTH

In determining the strength of a structure, three fundamental theories are used: the laws of statics, the elastic theory, and the ultimate theory. Since the average stresses in a solid prestressed lift slabs are generally low, perhaps on the order of 350 psi, their calculation is not of prime significance, except when used empirically as a measure of strength and deflection. It is because of this that allowable tensile stresses cannot be fixed but must be adjusted to the layout and loading conditions. However, by using suitable allowable stresses, sufficient strength can usually be obtained.

DESIGN FOR PROPER BEHAVIOR

Proper behavior means the limitation of cracks, the provision for creep and shrinkage and, above all, the absence of excessive camber, deflections, cracks, and vibrations.

The deflections in a prestressed slab are predicted by the elastic theory. However, the mathematics involved in the elastic theory for slabs is quite complicated, especially when prestressing is included. It is possible to simplify the problem by taking other approaches, but it has been found that accurate optimum solutions may be obtained with the help of electronic computers.

Also, to achieve proper behavior, one must work with the concept of actual live loads. It is possible in a prestressed lift slab to reach practically zero deflection for a given loading condition; however, zero deflection cannot be achieved for all loading conditions, nor for even one real loading condition if assumed loads and stresses are incompatible. If slabs are to behave properly, there must be no unrealistic live loads, impossible loading conditions, and empirically fixed allowable stresses.

APPLICATION IN BUILDING DESIGN

To illustrate, consider the case of an office floor. It is quite possible that for over 90 percent of its life, it will experience a superimposed load of only about 10 psf. This is the real load the slab should carry with zero deflection. The full superimposed load, of 50 psf, as determined by code requirements, must also be considered, and the slab must be designed to provide ample factor of safety for possible overload. But to design the slab to be flat under the full 50 psf of superimposed load, so much prestress will have to be added that the slab will camber up excessively under normal loading conditions. This camber would grow with time, as creep takes place in the concrete. Thus, a totally unsatis-

factory structure would result from such a design, while its capacity to carry overload is unnecessarily high.

Certainly a more rational way to design such a floor is to assume a reasonable live load of 10 psf, and use this load for the zero or near-zero deflection condition. To carry the full live load, the slab must have sufficient stiffness and sufficient ultimate strength to yield a proper factor of safety. To make both loadings compatible with proper behavior of the slab, it is often necessary to allow a certain amount of tensile stress under the full design load.

The amount of tension must be varied with many factors, keeping in mind that tension is always allowed in reinforced concrete. It is not suggested that undue liberty be taken with design criteria, but the importance of a rational approach as an important step to successful deflection control cannot be overemphasized.

Architecturally speaking, a flat slab is always desirable. Wavy cantilevers are an eyesore; deflected slabs making fitting of partitions, windows, etc., difficult and expensive; in the case of a deflected roof slab, drainage problems may arise.

Structurally speaking, the importance of a flat slab lies in its ability to withstand creep. A slab which cambers up under its own dead load signifies that its bottom fibers are under greater compression than its top fibers. The bottom fibers will shorten due to creep and the upward deflections will perhaps double or triple in time. If, on the other hand, the deflection is zero, it is then not affected by creep since there will be only axial shortening. Thus, practical field considerations such as small differences in mix proportions, effects of placing and vibration, variations in the values of the moduli of elasticity, small errors in the magnitude of the prestressing force, become much less significant as far as the behavior of the slab is concerned.

A further item of significance is the fact that a slab which is under axial stress has great strength to resist stress reversals. This is an important item, as it means that the slab has the ability to withstand vibrations, stresses due to unbalanced loading, earthquake forces, settlement, etc.

DESIGN EXAMPLE

For a design example, consider the slab shown in Fig. 3. It represents an apartment building floor.

The first problem, of course, is that of layout. Rarely is the design engineer the one who determines the dimensions of the building, yet the only way to achieve an economical structure is to make its arrangement entirely compatible with the type of construction chosen. However, a prestressed lift slab, by virtue of its high strength, gives greater flexi-

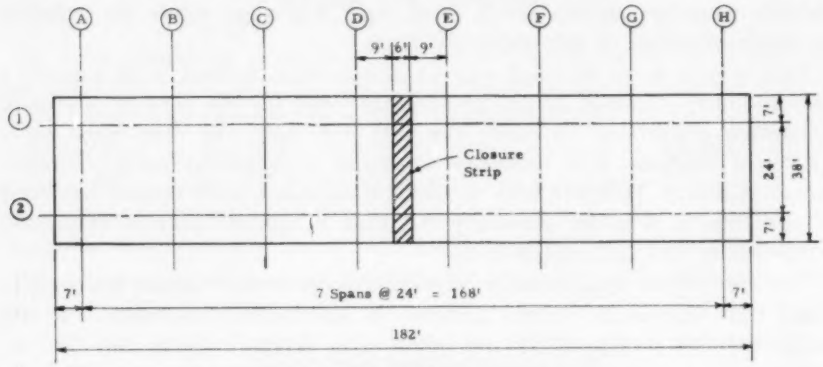


Fig. 3—Slab layout

bility of layout than other types of construction. It is important to select span and cantilever dimensions in their proper proportions, locate openings where they will not require costly framing, and choose the optimum slab thickness.

In the slab under consideration, spans are chosen to be 24 ft, with 7 ft cantilevers. These span dimensions fit the average apartment building well, but they may have to be varied depending on architectural requirements. Spans up to 30 ft are entirely feasible for solid flat slabs; longer spans may require waffle slabs or other structural configurations.

Cantilevers are desirable, even though not absolutely essential. They help to reduce moments in the adjacent spans and make for greater shear resistance around the column.

The slab thickness in this building is 7 in., which gives a depth to span ratio of 1 /41. This is acceptable for solid slabs, but too high for waffle type slabs. Also, in roof slabs, where vibrations are relatively inconsequential, the span to depth ratio can be higher than in floor slabs, where rigidity of slabs is quite essential.

Materials

The concrete strength used for prestressed lift slabs is usually 4000 psi at 28 days and 3500 psi at transfer of prestress. Generally this strength is not required for the low stresses under design loading, but the higher strength concrete does give better creep-resisting qualities, has a greater modulus of rupture and can withstand higher bearing pressures under bearing plates.

The ultimate steel strength should be 240,000 psi minimum, and elongations should be 3 percent in a 10-in. sample. The type of tendon used will vary, but the size of the tendons should be small enough to permit curvature to the desired cable profile.

Mild steel used is the ordinary ASTM A 305 used in conventionally reinforced concrete.

Loading

As was mentioned before, the loading used for design must be chosen logically. In the case under consideration, an apartment building, many codes call for a live load of 40 psf; 10 psf of miscellaneous loading is usually added. This floor will be loaded with a superimposed load of 10 psf most of the time, and full live load, while possible, will be the exception rather than the rule. Therefore, the floor will be designed for the following conditions:

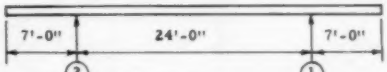
1. Girder load (7 in. slab) + 10 psf = 87.5 + 10.0 = 97.5 psf. Slab with no deflection under this loading. (Note: Under certain conditions a small deflection may be permissible.)

2. Full dead load and live load = 87.5 + 40.0 + 10.0 = 137.5 psf. This loading with some deflections and permissible tension.

If partition and ceiling loads are required, they, too, must be considered in the design. This applies equally to special conditions, such as concentrated loads due to stairways, heavy equipment loading, etc.

Analysis

In the beam method, which is most convenient and accurate enough for a case such as this, the slab is considered acting as a continuous beam with full load carried in each of two directions. The stresses which this analysis yields are not real, but with proper adjustment of allowable stresses, the design can be made entirely conservative. Maximum compressive stresses hardly ever enter the picture, since the average compression in the slab is almost always under 500 psi. An important aspect is the allowable tensile stress, which may be permitted under full dead load and live load. There is no danger in permitting a tensile stress to exist in a prestressed structure, but its accepted magnitude should be adjusted according to a number of factors, including the cross-sectional shape of the slab. Tensile stresses of 0.04 f_c' or higher are accepted for solid slabs, for ribbed slabs this should generally not exceed 0.02 f_c'. Higher tension may be permitted in concrete at transfer.



M_{GL}	-2.14	+4.16	-2.14
M_{DL}	-2.40	+4.60	-2.40
M_{DL+LL}	-3.40	+6.50	-3.40

Fig. 4 — Moments in transverse direction (per foot of slab width). Note: All moments in kip-feet

Stress in steel at anchorage is usually on the order of 150,000 psi, even though the steel may be tensioned as high as 0.80 f_s' for short periods to overcome stressing friction. The effective stress in the steel is computed by deducting the losses due to elastic shortening, creep, and shrinkage from the steel stress at anchorage. These losses, not including friction, are generally lower than in prestressed beams and may be set at 10-15 percent, depending on the average stress in the slab.

Cable profiles

The actual selection of the cable profiles and prestressing forces is largely a matter of trial and error. Specifically, a cable profile must be chosen, from which stresses and deflections are computed. The results

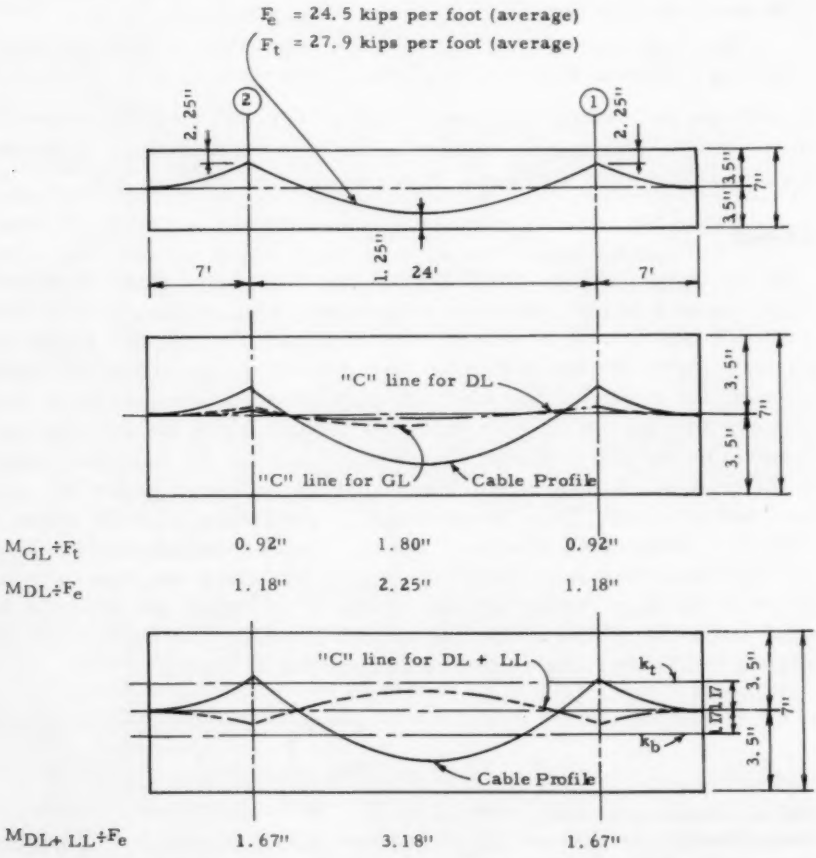


Fig. 5—Cable profile and *C* line in transverse direction

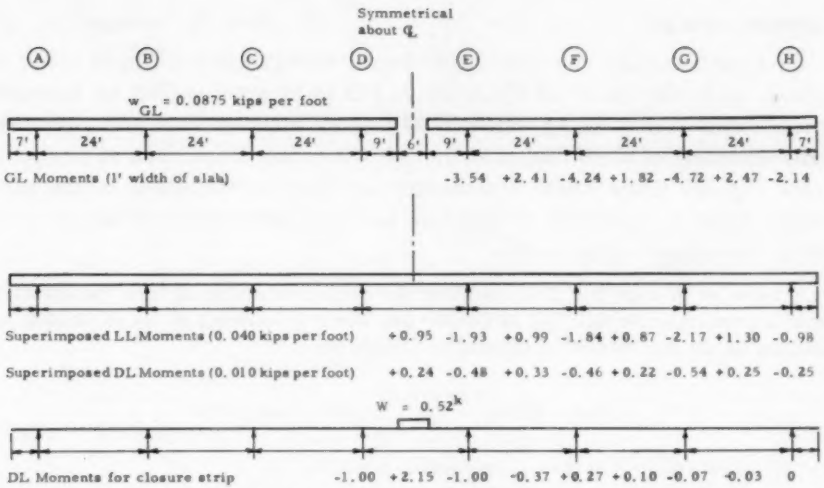

Symmetrical about ℄
A B C D E F G H
w_{GL} = 0.0875 kips per foot
7' 24' 24' 24' 9' 6' 9' 24' 24' 24' 7'
GL Moments (1' width of slab) -3.54 +2.41 -4.24 +1.82 -4.72 +2.47 -2.14
Superimposed LL Moments (0.040 kips per foot) +0.95 -1.93 +0.99 -1.84 +0.87 -2.17 +1.30 -0.98
Superimposed DL Moments (0.010 kips per foot) +0.24 -0.48 +0.33 -0.46 +0.22 -0.54 +0.25 -0.25
W = 0.52k
DL Moments for closure strip -1.00 +2.15 -1.00 -0.37 +0.27 +0.10 -0.07 -0.03 0

Fig. 6—Moments in longitudinal direction (per foot of slab width). Note: All moments in kip-feet

can be best shown graphically by the plotting of the *C* line,[5] which is the line of the resultant compressive force in the concrete section. It follows that when the *C* line for any given loading coincides with the neutral axis of the concrete section, the section is under axial compression, hence has no deflection. If the *C* line stays within the kern points of the section, no tension exists under the particular loading.

For the design in the transverse direction, moments are shown in Fig. 4. These were obtained by considering a 1-ft strip of slab, analyzed as a continuous beam. No linear transformation of the cable profile is involved in this section, so that the trial and error procedure is rather simple. Resulting cable profiles along with *C* lines are shown in Fig. 5. (For simplicity the curve of the cable may be assumed to come to a point at the column lines, even though the actual cable profile will have a reverse curve. The error introduced will be on the order of 5 percent or less.) It will be noted that the *C* line for girder load falls close to the neutral axis, which means that shrinkage and creep will be largely axial. On that basis, then, the *C* line for actual dead load is plotted. Again, it follows the neutral axis. *C* lines for dead load and live load show some deflection, but they fall within the kern points, denoting absence of tension under maximum loading.

The design process is repeated for the longitudinal direction. This time linear transformation must be considered.[5] Moments are shown in Fig. 6, actual and transformed cable locations as well as *C* lines in Fig. 7. Tension is found at column lines *D* and *E*, amounting to 120 psi, which under full dead load and live load is 0.30 f_c' and is considered satisfactory.

Ultimate strength

In a continuous structure, the ultimate strength is a difficult thing to check, since the theory of limit design has to be applied. But an approximate solution may be obtained by taking an interior span and assuming the formation of yield hinges at the column center lines, then at midpoint (see Fig. 8). Since linear transformation may be neglected in the ultimate range, it is possible to ascertain the ultimate moment at any section from the actual cable profile.

A force of 21 kips per ft is required for elastic behavior. It may be assumed that a greased cable will fail at 210,000 psi. Using a working stress of 135,000 psi and for an ultimate stress 90 percent of 210,000 psi

$$F_{ult} = 21.0 \times \frac{210 \times 0.90}{135} = 29.4 \text{ kips per ft}$$

By Whitney's theory[5] at the column center line (Fig. 9)

$$k'd = \frac{29{,}400}{0.85 \times 4000 \times 12} = 0.72 \text{ in.}$$

$$a = 7.0 - 1.25 - \frac{0.72}{2} = 5.39 \text{ in.}$$

$$M_{ult} = \frac{29.4 \times 5.39}{12} = 13.2 \text{ kip-ft}$$

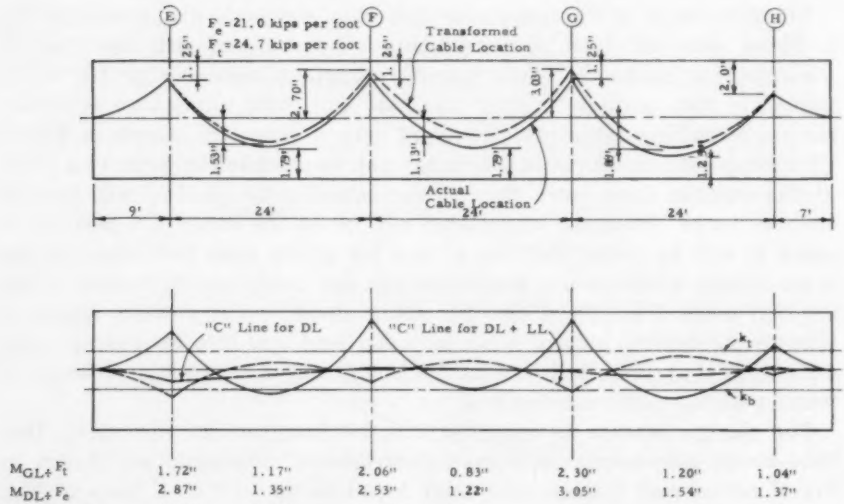


Fig. 7—Cable profiles and *C* lines in longitudinal direction

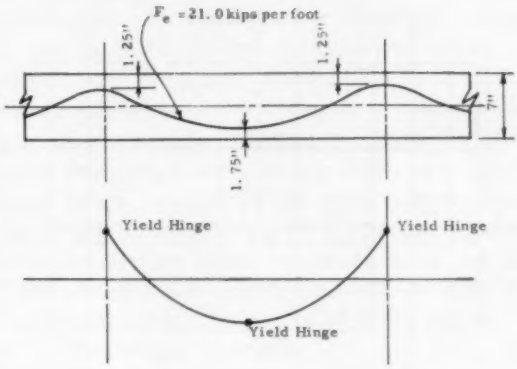


Fig. 8 — Location of yield hinges

Similarly for midspan

$$k'd = 0.72 \text{ in.}$$

$$a = 7 - 1.75 - \frac{0.72}{2} = 4.89 \text{ in.}$$

$$M_{ult} = \frac{29.4 \times 4.89}{12} = 12.0 \text{ kip-ft}$$

The sum of the M_{ult} must equal $wL^2/8$ hence

$$13.2 + 12.0 = \frac{wL^2}{8} = \frac{w(24)^2}{8}$$

$$w_{ult} = 350 \text{ psf}$$

Dead load plus maximum live load is 137.5 psi; therefore, the load factor is 350/137.5 = 2.55. This agrees well with the load factor of 2.4 required by the ACI-ASCE recommendations.[6]

With these computations the cables may now be laid out in the slab. The total force per bay is distributed: 60 percent to column strip and 40

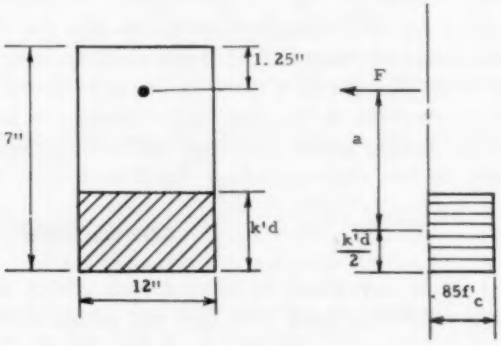


Fig. 9—Stress block for ultimate range

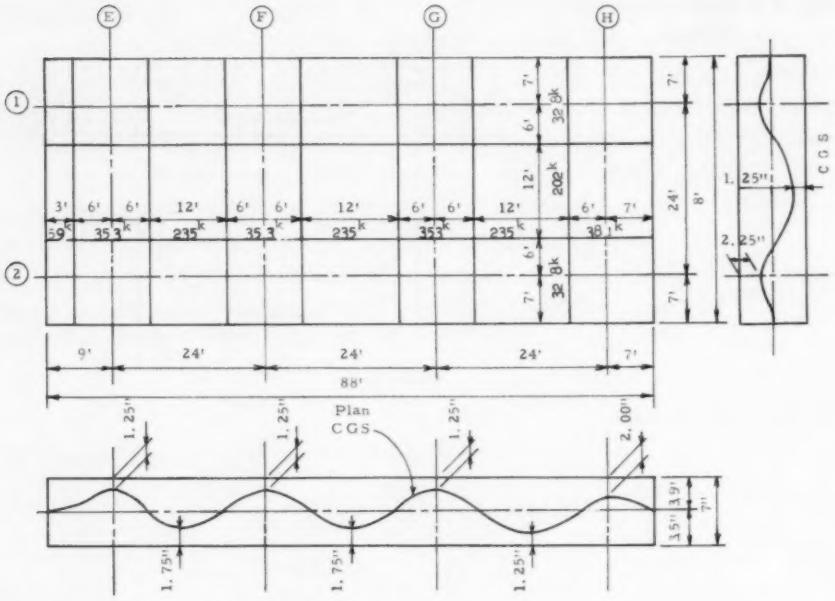


Fig. 10—Final arrangement

percent to middle strip. This distribution is empirical, based on test data and field observations. The final forces and profiles are shown in Fig. 10.

Other design features

With the forces and profiles now known, (Fig. 10), the slab design may be finished by computing shears in the slab, designing collars, columns, connections, etc. The design of the collar is usually governed by punching shear calculations.[7] It should be pointed out that an important feature is the connections to walls and columns which must take account of shrinkage, creep, and movement of slabs.

Perhaps the most important aspect is the choice of slab thickness. Certain rules of thumb have been used, such as depth to span ratio of 1/36 or 1/45. These ratios may serve as a guide but must be used with extreme caution. A thin slab will vibrate and is difficult to make flat. However, making a slab too thick will often require too much prestress, and may be too rigid to adapt itself to lifting operations.

CONCLUSIONS

1. It is important to have a slab which shows little or no deflection under its own dead load and the actual sustained live load. This is not

only for architectural reasons, but also to minimize the effects of creep, to provide greater strength, and to allow for construction tolerance.

2. Deflections may be computed by the elastic theory. The load capacity is determined by the ultimate theory.

3. The beam method of analysis, whereby the slab is divided into a series of beams, running on a 90 deg grid, is convenient. While not exact, it is often sufficiently accurate when used with the proper judgment. More exact optimum solutions can be obtained with electronic computers.

4. Design loading and allowable stresses must be varied from job to job. Basic understanding of the structural behavior of slabs, as well as seasoned experience, must be used to guide the choice of these values.

5. Care must be exercised in the design of waffle and joist slabs, as well as of slabs of unusual layouts and proportions.

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Title No. 56-41

Behavior and Strength in Shear of Beams and Frames Without Web Reinforcement

By ROGER DIAZ DE COSSIO and CHESTER P. SIESS

Forty-nine simply supported beams and 24 frame members were tested under several different types of loading. These included: two-point loads, midspan concentrated loads, uniform load, and axial load. The frame members and 18 simple supported beams were tested under uniform load. Ten of the beams under midspan concentrated loads were tested with an axial load of 20 kips. Special emphasis was placed on the effects of axial and uniform loads on shear strength and behavior of the members. Other variables were concrete strength, steel percentage, and span length.

The test results are given, discussed, and correlated. The use of the inclined tension cracking load as a measure of the useful shear strength of the members is advocated, and a procedure is given for its determination.

■ THE RESULTS OF TESTS on 49 simply supported beams and 24 frame members are described, discussed and correlated. The main object of these tests was to study the behavior and strength in shear of reinforced concrete members. The ultimate objective of the test program was to obtain information that would permit the development of more rational design criteria for reinforced concrete box culverts under deep fills. Owing to the characteristics of reinforced concrete box culverts, the greater emphasis was placed on members under axial and uniform loads, and without web reinforcement. However, three of the frame members had web reinforcement in the form of bent bars.

The major variables studied included type of loading, concrete strength, steel percentage, ratio of span length to effective depth, ratio of shear span to effective depth, and ratio of axial load to vertical load. The simply supported beams were tested under two-point loads; under one

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midspan concentrated load applied through an integrally cast column stub, with and without axial load; and under uniform load. The frame members were tested under uniform load. The uniform load was simulated in all cases by ten closely spaced loads applied by hydraulic jacks.

DEFINITIONS AND NOTATIONS

Inclined tension crack is a diagonal crack that results from a combination of bending and shearing stresses; it may be the source of an eventual shear failure.

Fully developed inclined tension crack is an inclined crack that causes a sudden redistribution of steel strains or complete collapse of the member.

Cracking load is the load at the formation of the first fully developed inclined crack.

Diagonal tension failure is a failure occurring simultaneously with the sudden appearance of the first fully developed inclined tension crack, or at the cracking load.

Shear-compression failure is a mode of failure in which the specimen is able to carry additional load after the formation of the first fully developed inclined tension crack.

Flexural tension failure occurs initially due to yielding of the tensile reinforcement in the regions of high moment, and final collapse occurs due to crushing of the concrete in the compression zone after considerable deformations have taken place.

Critical section is a section that corresponds roughly to the point in a member where a fully developed inclined tension crack crosses middepth.

The following notation has been used in this paper:

a = shear span in a simply supported beam, the distance between the support and the nearest load

a_1 = distance between column face section and the point where a 45-deg bent bar crosses middepth in a frame member

$A = 2.14(\text{lb})^{1/2}(\text{in.})^{-1}$ (Eq. 1)

A_s = area of tensile steel, or of positive steel in a frame member

$A_{s'}$ = area of negative steel in a frame member

B = 4600 psi (Eq. 1)

b = width of member

C = internal compressive force in the concrete at ultimate flexural capacity

c = ratio of axial to vertical load in a frame member

d = effective depth of reinforcement

$D = 1 + 0.04\left(\frac{N}{V}\right)_x$

f_c' = compressive strength of concrete as determined from 6 x 12-in. control cylinders

f_r' = modulus of rupture of concrete as determined from 6 x 6 x 18-in. control beams

f_y = yield point stress of tensile or positive reinforcement

f_y' = yield point stress of negative reinforcement

h = total depth of member

k_1 = ratio of area of concrete stress block to area of enclosing rectangle

k_2 = fraction of depth of compressive zone in concrete which determines the position of the compressive force C in the concrete

k_3 = ratio of maximum compressive stress in concrete to cylinder strength

k_u = ratio of depth of compressive zone to effective depth

L = over-all length of frame member

l = clear span length of frame member

l' = simply supported span length; for frame members l' = span between points of contraflexure = $l\sqrt{M_R}$

λ = fraction of the effective depth that determines the position of the axial load

M = bending moment

M_x = bending moment at a particular location, at cracking load

M_f = ultimate flexural resisting moment

M_o = ultimate flexural resisting moment when no axial load is present

M_R = moment ratio; the ratio of maximum positive moment to the total moment between column face sections in a frame member

N = axial load in a member; when followed by subscript x is to be considered at that particular location

N_c = axial load at first inclined tension cracking load

N_u = axial load at ultimate capacity

$p = A_s/bd$

$p' = A_s'/bd$

$q = pf_y/f_c'$

T = tension force in steel at yielding $= pbdf_y$

V = shear

V_x = shear at a particular location, at cracking load

W = total vertical load

W_c = total vertical cracking load

W_u = total vertical failure load

W_f = measured or computed flexural failure load

$\left(\frac{N}{V}\right)_x$ = ratio of axial load to shear at a particular location, at cracking load; at the point of contraflexure $N/V = cL/2l'$

$\left(\frac{Vd}{M}\right)_x$ = ratio of shear to moment multiplied by the effective depth, at a particular location, at cracking load.

SIMPLY SUPPORTED BEAMS

Test Program

All beams had a 6 x 12-in. cross section, and were reinforced in tension only. The tests can be divided into three different series: (a) beams under two-point load; (b) beams loaded through an unreinforced column stub at midspan, with and without axial load; and (c) beams under uniform load. Most of the beams

TABLE 1—PROPERTIES OF BEAMS

Mark	Shear span a, in.	Span length l', in.	Yield point stress f_y, ksi†	Concrete strength f'_c, psi‡	Modulus of rupture f_r, psi‡
(a) Beams under two-point load $b = 6$ in., $d = 9.94$ in., $h = 12$ in. Reinforcement: 2-#9 bars; $p = 3.36$ percent					
L-1*	20	76	44.0	3050	350
L-2*	30	96	45.0	3120	333
L-2a	30	96	41.0	5320	583
L-3	40	116	45.0	4060	467
L-4	50	136	44.0	3740	458
L-5	60	156	48.0	4050	583
L-6*	70	176	46.0	4440	550
Retests					
L-1R	20	40	44.0	3050§	350§
L-2R*	30	60	45.0	3120§	333§
L-2aR	30	60	41.0	5230	583§
L-3R	40	80	45.0	4060§	467§
(b) Stub beams $b = 6$ in., $d = 10$ in., $h = 12$ in. Reinforcement: 3-#4 bars; $p = 1.0$ percent					
No axial load					
A-1	20	52	66.5	4070	417
A-2*	30	72	68.0	4570	550
A-3	40	92	65.6	2820	467
A-4	50	112	66.6	3890	642
A-5	60	132	67.0	4450	383
Axial load 20 kips					
B-1	20	52	66.5	3780	458
B-2	30	72	66.5	4140	475
B-3	40	92	57.1	3820	475
B-4	50	112	66.5	4100	458
B-5	60	132	67.0	4120	542
Reinforcement: 2-#9 bars; $p = 3.33$ percent					
No axial load					
A-11	20	52	49.5	4100	417
A-12	30	72	45.5	3870	483
A-13	40	92	57.0	3210	425
A-14	50	112	52.8	3990	458
A-15	60	132	48.1	3630	450
Axial load 20 kips					
B-11	20	52	48.1	3660	450
B-12	30	72	56.8	3930	458
B-13	40	92	51.4	4050	408
B-14	50	112	52.6	4250	433
B-15	60	132	47.3	4110	468
(c) Beams under uniform load $b = 6$ in., $d = 9.94$ in., (10⅝ in. for Beam D-5), $h = 12$ in. Reinforcement: 3-#4 bars; $p = 1.01$ percent					
D-15		88	66.8	3860	483
D-14		88	67.0	4650	425
D-16		88	85.0	5760	450
D-13		110	67.3	2780	442
D-12		110	67.0	4160	508
D-17		110	85.0	5970	483
Reinforcement: 2-#6 bars; $p = 1.35$ percent					
D-5*		66	52.3	3740	425
Reinforcement: 3-#6 bars; $p = 2.21$ percent					
D-4		88	44.3	5020	592

TABLE 1 (Continued)—PROPERTIES OF BEAMS

Mark	Shear span a, in.	Span length l', in.	Yield point stress f_y, ksi†	Concrete strength f_c', psi‡	Modulus of rupture f_r', psi‡
(c) Beams under uniform load $b = 6$ in., $d = 9.94$ in., $h = 12$ in. Reinforcement 3-#4 bars; $p = 1.01$ percent					
D-9		88	42.0	2790	383
D-10		88	39.0	3540	458
D-11		88	48.3	5840	525
D-7		110	42.0	2700	458
D-6		110	45.0	3450	442
D-1*		110	42.8	4470	516
D-2		110	44.5	5590	483
D-8		110	57.2	6600	767
D-3		132	45.5	4820	567
D-18		154	49.5	4240	480

*Beams had no clamp-on stirrups at the ends.
†Average yield point stress of the bars used in the beam.
‡Average values obatined from control cylinders and beams cast and cured together with the corresponding beam, and tested the same day the beam test was performed.
‖Values obtained at the time of the original tests.

had clamp-on external stirrups at the end of the spans to prevent splitting along the reinforcement. The properties of all the beams are listed in Table 1.

Beams under two-point load—Seven beams, designated L-1 through L-6, were tested in this series. The two loads were symmetrically placed about midspan, always 36 in. apart. All beams had a tension reinforcement ratio of 3.36 percent, and an effective depth of 9.94 in. The main object of this series of tests was to study the effect of the moment-shear ratio on mode of failure; more specifically, the value of the moment-shear ratio for which the mode of failure changed from shear to flexure was investigated for one basic cross section. Since, for the case of a beam under one or two symmetrical concentrated loads, the ratio of moment to shear is equal to the distance between the support and the first load, that is, to the shear span, this distance was varied systematically from 20 to 70 in. Four of the beams that failed in shear at one end were again tested at the other end, using the same shear span as before, but under one concentrated load at midspan; thus one half of the beam consisted of what originally was a pure moment region, while the other half remained unchanged from the original arrangement. The two different setups can be seen in Fig. 1. The retests were designated L-1R through L-3R, corresponding to the original beams marked L-1 through L-3.

Stub beams with and without axial load—The object of these tests was to investigate the effect of axial load on behavior and strength in shear, for different

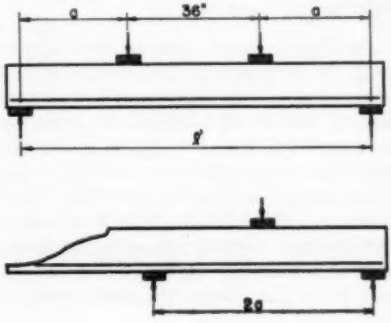

Fig. 1—Details of beams under two-point loads and retests

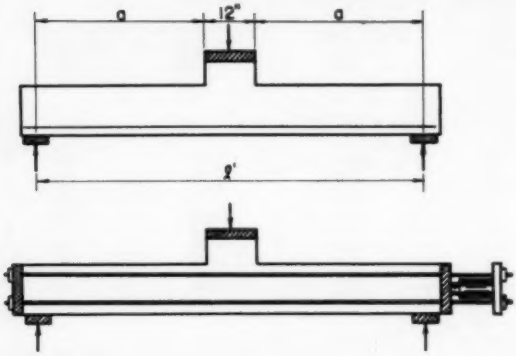

Fig. 2—Stub beams with and without axial load

values of the moment-shear ratio. Twenty beams were tested in pairs, one without axial load and one with an axial load of 20 kips, applied at middepth, which was kept constant throughout the tests. Two different steel ratios (1.0 and 3.33 percent), and five span lengths (52 to 132 in.), or moment-shear ratios, were included in this series. The axial load was applied by means of a hydraulic jack bearing against one end of the beam and connected to the other end by four steel rods running along the sides. The vertical load was applied also by means of hydraulic jacks acting through a bearing plate on the column stub. The test arrangements with and without axial load are shown in Fig. 2. Beams under no axial load were marked A-1 through A-5 for $p = 1.0$ percent, and A-11 through A-15 for $p = 3.33$ percent. Beams under an axial load of 20 kips were marked B-1 through B-5 for $p = 1.0$ percent, and B-11 through B-15 for $p = 3.33$ percent. Concrete strength was not a primary variable in this series.

Beams under uniform load—These tests were designed to investigate the behavior and strength in shear under uniform load of simply supported rectangular beams without web reinforcement. The major variables included in this series were: ratio of simply supported span length to depth (from 6.1 to 15.5); steel ratio (from 1.01 to 3.36 percent); and concrete strength (from 2800 to 6600 psi). A total of 18 beams, designated D-1 through D-18 were tested. The uniform load was simulated by ten equally spaced hydraulic jacks acting through 6 in. long bearing blocks at the top of the beam. The arrangement, shown in Fig. 3, simulates closely a uniform load. To insure shear failures, high yield point steel was used for the beams with low steel ratios.

Fabrication and curing

All beams were cast in steel forms with adjustable end plates, in the same position as tested. For all beams on which mechanical strain readings were to be taken, gage lines were marked on one outside bar, and holes were punched and drilled. Corks of 1 3/8 in. diameter were wired to the bar over the gage holes, to provide access for strain measurements after casting. The steel was held in position at the bottom of the form by chairs of appropriate dimensions. Two or three pieces of 3/4-in. pipe were distributed along the beam as spacers. The concrete was mixed from 2 to 8 min in a 6 cu ft capacity nontilting mixer. One or two batches of concrete were necessary for each beam. The concrete was placed in two horizontal layers along the beam. At least one 6 x 20-in. control beam and three 6 x 12-in. cylinders were made from each batch. All of the concrete

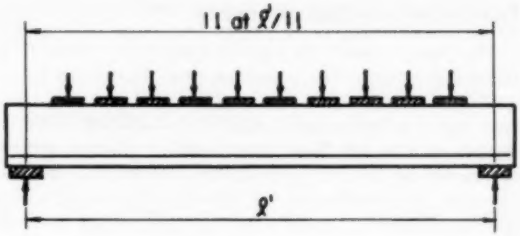

Fig. 3—Beams under uniform load

was placed in the previously oiled forms with the aid of a high frequency internal vibrator. For the stub beams, a steel attachment of 6 x 6 x 12-in. internal dimensions was placed at the top of the form, so that the stub could be integrally cast with the beam. Several hours after casting, the top surface of the beams was troweled smooth, and all cylinders were capped with neat cement paste. The beams and control cylinders were removed from the forms the day after casting and stored under moist conditions for 6 to 8 days. They were then stored in the laboratory until tested.

Materials

Concrete was made with standard brands of Type I portland cement, and Wabash River sand and gravel, all purchased in several lots from a local dealer. They all passed the usual specifications tests. The coarse aggregate had a maximum size of about 1 in., and contained a relatively large percentage of fines, with a fineness modulus that varied for the different lots from 6.5 to 7.0. The fineness modulus of the sand varied from 2.7 to 3.2. Absorption in both aggregates was about 1 percent by weight of saturated surface dry aggregates. Since all mixes were based on results obtained in former investigations in the laboratory using the same type of aggregates, it was possible to obtain actual cylinder strengths in reasonably good agreement with the nominal strengths. Average values obtained from control cylinders and flexure specimens are shown in Table 1 for the corresponding beams.

Deformed bars meeting the requirements of ASTM A 305 were used in all beams. The bars were usually delivered in 22 ft lengths. One coupon 2 ft long was cut from each bar and tested in tension. Stress-strain curves were obtained in about one-third of the tension tests. Three different types of steel were used for the simply supported beams, all having a definitely flat yield point region. For beams with small steel percentages, high strength steel was used to insure shear failures. The special hard grade steel used in Beams D-16 and D-17 had an average yield point of 85,000 psi, an ultimate strength of 137,500 psi, and an average modulus of elasticity of 24,300,000 psi. For beams with larger steel ratios or shorter lengths, as shown in Table 1, the steel used had yield points ranging from 66,500 to 68,000 psi, with an average ultimate strength of about 121,000 psi, and a modulus of elasticity of about 27,400,000 psi. Finally, intermediate grade steel was used for all other beams, with yield point stresses ranging from 39,900 to 57,200 psi, an average ultimate strength of about 75,300 psi, and an average modulus of elasticity of 29,600,000 psi. The bars used in each beam were matched as closely as possible on the basis of their yield point strengths. The value of the average yield point stress for the reinforcement in each of the beams is listed in Table 1.

Equipment and test procedure

The beams under two-point load were tested in a 200,000 lb capacity screw-type testing machine. The machine was used both to apply deformation and to measure load. The machine was calibrated prior to use in the investigation, and satisfied the requirements of ASTM E 4-47T. The load was applied through steel balls seated on 6 x 6 x 2-in. steel bearing blocks, either set on the beam with plaster, or seated on leather pads 3/8 in. thick. The leather pads were necessary in some cases to provide space for SR-4 strain gages attached to the top of the beam at the longitudinal center line. All the beams were supported through 6 x 6 x 2-in. bearing blocks attached with plaster to the bottom of the beam on a 2-in. half-round at one end, and a 2-in. roller at the other.

Both the stub beams and the beams under uniform load were tested in a special steel testing frame with the loads applied by hydraulic jacks. The jacks were connected by high pressure hoses to a brass manifold which in turn was connected to a pressure gage and a hydraulic pump. The jacks-manifold-hydraulic pump arrangement was calibrated prior to use in the investigation by a testing machine, so that total load was read directly on the pressure gage. Ten 10-ton jacks were used for the tests of beams under uniform load, and four were used for the stub beams. Because of the nature of the hydraulic system, the load on each jack was the same at any one time, regardless of the ram extension, except for negligible differences due to friction. The jacks were held with their bases against the steel reaction beam, and the jack rams moved down against the beam during testing. The loads were transmitted to the bearing blocks through 1.5-in. diameter steel balls. The beam was separated from the bearing blocks by 3/8 in. thick leather pads. This arrangement facilitated the positioning of the jacks with respect to the beam. The support arrangements were the same as previously described for the beams under two-point load. The support bearing blocks rested on the laboratory floor through concrete abutments.

In some of the beams under two-point load, measurements were made of the deformations at the top fiber of the beams by means of SR-4 electrical resistance strain gages. Deflections were measured in all beams at midspan with 0.001-in. dial indicators mounted on posts that were in turn attached to a deflection frame. In some of the beams, deflections were also measured beneath the load points.

Steel deformations were measured mechanically in some of the beams under uniform load and stub beams. Strains were measured on one side only, over contiguous 6-in. gage lengths. In this way, continuous steel strain distributions at different load levels were obtained. Except for the very short beams, deflections were measured at both midspan and the quarter points.

For some of the beams under uniform load, to determine the horizontal strain distribution in a vertical section through an inclined crack, electrical strain gages were placed at that section. Since the exact position of the inclined crack could not be known in advance, the strain gages were applied after the inclined crack had appeared. To do this, the load was removed, the strain gages applied, allowed to dry for a day or so, and then the load at which the crack had formed was again reached in five or six quick increments.

The loads were applied in increments until complete collapse occurred, or until the beams carried no more load under increasing deformation. The number of increments to failure varied from 8 to 22. After each increment of load was applied, deflections and strain readings were taken, and cracks were observed through a low-power illuminating magnifying glass, and marked with ink. There was usually some drop off in load and some increase in deflection while readings

were being taken. These changes were recorded before the next load increment was applied. Photographs of the test beams were taken at important stages of loading, and after failure.

TEST RESULTS

Presentation of test data

Results of the tests are summarized in Table 2, which includes loads and midspan deformations at the formation of the first fully developed inclined tension crack and at failure of the beams. The failure loads are the maximum that the beams were able to carry. Sometimes, due to the nature of the loading equipment, the load decreased slowly after the maximum; at other times the load dropped suddenly to a very low value. The steel strains at failure were extrapolated from plots of steel strain versus deflection and steel strain versus load. It is believed that these strains are accurate to within 10 to 20 percent, and they are intended only as a guide to the behavior of the beams. In a few cases, owing to the suddenness of the failure, it was not possible to record the deflections. In all such cases, the deflections were also extrapolated from load-deflection curves. The last column of Table 2 lists the modes of failure of the beams tested. Whenever more than one symbol is used, it means that the failure occurred in both modes simultaneously, or almost simultaneously. The load values include live load only, since the weight of the beam was never more than about 2 or 3 percent of the inclined crack-

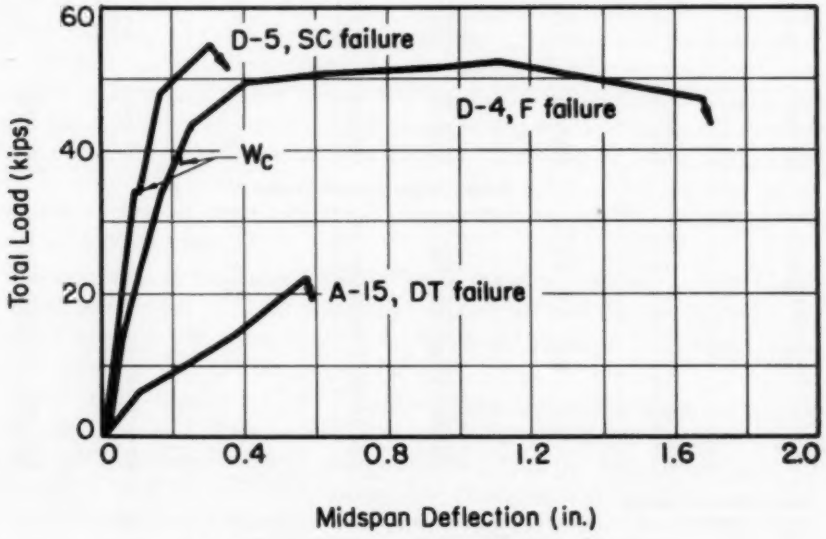

Fig. 4—Typical load-deflection curves for beams

TABLE 2—TEST RESULTS

Mark	Inclined tension cracking			Failure			Mode of failure†
	Total load W_c, kips	Midspan steel strain × 10^6	Midspan deflection, in.	Total load W_u, kips	Midspan strain* × 10^6	Midspan deflection, in.	
(a) Beams under two-point load							
L-1	30.0		0.264	52.2		0.598	SC
L-2	26.0		0.229	34.0		0.434	SC
L-2a	28.7		0.234	36.0		0.371	SC
L-3	24.0		0.267	24.0		0.267	DT
L-4	23.0		0.544	23.0		0.544	DT
L-5	22.8		0.825	22.9		0.825	SC, DT
L-6	—		—	21.1		1.330	F
Retests							
L-1R	74.0		0.360	74.0		0.360	DT
L-2R	27.4		0.085	33.6		0.160	SC
L-2aR	31.8		0.090	41.6		0.172	SC
L-3R	27.8		0.125	27.9		0.195	SC, DT
(b) Stub beams No axial load							
A-1	20.6	128—	0.068	33.0	174 +	0.158	SC
A-2	18.8	88 +	0.224	18.8	88 +	0.224	DT
A-3	15.4		0.190*	15.4		0.190*	DT
A-4	15.8		0.561	15.8		0.561	DT
A-5	—		—	14.7		2.028	F
Axial load = 20 kips							
B-1	29.8		0.066	51.4		0.370*‡	SC
B-2	23.4		0.307	29.5		0.366	DT
B-3	20.6		0.309	21.8		0.350‡	SC
B-4	18.9		0.380*	18.9		0.380*	SC
B-5	—		—	15.1		0.920	F
No axial load							
A-11	28.3	51	0.046	46.5	84	0.114	SC
A-12	26.5	64	0.099	26.5	64	0.099	DT
A-13	21.1	82*	0.110	21.1	82	0.110	DT
A-14	24.6	112*	0.374	24.6	112	0.374	DT
A-15	22.2	148*	0.574	22.2	148	0.574	DT
Axial load = 20 kips							
B-11	37.8	45	0.052	64.5	110	0.246	SC
B-12	29.6	60	0.129	33.7	76	0.217	SC
B-13	26.5	87—	0.212	27.8	161	0.385	SC
B-14	23.8	83	0.357	23.8	83	0.357	DT
B-15	21.1	93	0.530	21.1	93	0.530	DT
(c) Beams under uniform load							
D-15	28.0	200	0.290	38.8	780	0.911	SC
D-14	29.2	221	0.330	40.6	1375	1.250	SC
D-16	30.3	230	0.320	47.8	750	0.800	SC
D-13	22.8	220	0.613	24.5	241	0.733	SC
D-12	—	—	—	32.7	1630	2.077	F
D-17	24.7	230	0.569	29.8	282	0.760	SC
D-5	35.0		0.101	55.2		0.428	SC
D-4	39.8		0.215	52.6		1.014	SC, F
D-9	34.4		0.165	47.2		0.461	SC
D-10	38.0	100*	0.183	52.1	155	0.451	SC
D-11	48.2	68	0.201	71.3	193	0.469	SC
D-7	35.3		0.335	43.5		0.519	SC
D-6	35.3		0.306	46.8		0.504	SC
D-1	38.2		0.342	41.7		0.736	SC
D-2	37.6		0.282	49.6		0.583	SC
D-8	39.4		0.302	55.3		0.765	SC
D-3	38.8		0.535	42.3		0.919	SC
D-18	30.7		0.717	32.9		0.844	SC

*Extrapolated values.

‡Load-deflection curve indicated considerable yielding of the steel before failure.

†SC = shear-compression failure; DT = diagonal tension failure; F = flexural tension failure.

ing load. Typical load-deflection curves, steel strain distributions, and sketches of crack patterns, are shown in Fig. 4-7.

Behavior and modes of failure

Before appearance of the first flexural cracks, all of the beams exhibited an essentially elastic behavior; the deformations were proportional to the applied loads and an initial straight portion was observed in the load-deflection curves (Fig. 4). After flexural cracks developed in the regions of high moment, the behavior of the beams under increasing load varied according to their respective modes of failure. Three major modes of failure have been defined for the beams in this investigation: flexural tension, shear-compression, and diagonal tension (symbols F, SC, and DT, respectively, in Table 2).

In a flexural failure, bending stresses are predominant. In all the beams that failed in flexure, the initial vertical cracks developed considerably, and the failures were of a ductile character in that large deformations took place before final collapse. The beams tested were all under-reinforced; that is, flexural failure was due initially to yielding of the tensile reinforcement, and final collapse occurred by crushing of the concrete in the compression zone at midspan.

In the beams that failed in shear-compresssion, the flexural cracks developed up to a certain stage at which their progress was very slow, or stopped almost completely. At this stage, under further load, inclined cracks began to appear. Sometimes they were the continuation of existing flexural cracks; at other times they appeared slowly at first and near middepth, at sections away from that of maximum moment, and subsequently joined the existing flexural cracks. Under further load, the inclined cracks progressed upward toward midspan, and downward toward the tensile steel, producing splitting at the level of the steel toward the support. Final collapse occurred due to crushing of the compression zone at the top of a major inclined crack. This type of failure was more brittle in character than a flexural tension failure, and the deformations were seldom as large.

The behavior of the beams that failed in diagonal tension was identical to those that failed in shear-compression, except that the sudden appearance of an inclined crack caused complete collapse of the beams. This is also a sudden and brittle type of failure.

Some of the beams failed in combinations of the three major modes previously defined. Two beams (L-5 and L-3R), failed in a combination of shear-compression and diagonal tension; that is, they were able to carry little load beyond that causing the first inclined cracks. Beam D-4 was classed as a combination of shear-compression and flexure, because a definite inclined crack was formed before final collapse occurred by

crushing of the compression zone at midspan, after considerable deformation had taken place.

An inclined crack was considered to be fully developed when it progressed toward both midspan and the support and intersected the tensile steel. The load at which this stage was reached was denoted as the cracking load. The presence of a fully developed inclined crack induces a redistribution of steel stresses. The load-steel strain relationship is suddenly altered, since the inclined crack introduces additional deformations without a corresponding increase in load. In the beams tested, for loads smaller than the cracking load, the steel strain distribution had a shape similar to that of the moment diagram. However, when the cracking load was reached, the steel strain at the section of the inclined crack changed suddenly to a much higher value. This sudden change could not be observed in the beams that failed in diagonal tension, since by definition, the beam collapsed at the cracking load. The phenomenon of redistribution of steel stresses caused by an inclined crack has previously been discussed by other investigators,[1] and their observations agree with those of this paper. Typical steel strain distributions at load levels below, above, and at the cracking load are shown in Fig. 5.

The shorter beams failed in shear-compression, and the longer ones in either diagonal tension or flexural tension. For the shorter beams, the development of the inclined cracks was slow, and even with the help of steel strain measurements, the cracking load was of necessity a vaguely defined quantity. Two different observers might easily arrive at values of the cracking load differing by 10 or even 15 percent. But, no matter how vague it might be in some cases, the value of the cracking load is perhaps the most significant quantity associated with the shear failure of a beam, since it represents a definite change in the behavior of a member without web reinforcement. For longer members that fail in diagonal tension, the cracking load is the collapse load, a sudden and well-defined phenomenon. Its value is then determined with a good degree of precision.

There was evidence for some of the beams under uniform load that failure in shear-compression occurred after considerable yielding of the steel had taken place, and for some beams the strain hardening portion of the steel was reached. However, at the cracking load no strain above the yield point strain was measured.

In the retests of the beams under two-point load, the appearance of inclined cracks was sometimes difficult to detect, since they appeared over existing cracks that had already been marked in the original tests. However, the behavior was much the same as that described above. The inclined crack leading to the final collapse formed for some beams in the region that had been a pure moment region in the original tests, while

for others it formed in the shear span. Beam L-1R failed suddenly at an unusually high load, much greater than the computed flexural capacity, and it is possible that the shear and flexural strengths may have been increased by the restraints provided by the load and support bearing blocks, which for this case were relatively large compared to the span of only 40 in.

Crack patterns

In beams under two-point load, retests, and stub beams, the inclined crack formed always between the support and the load block or stub,

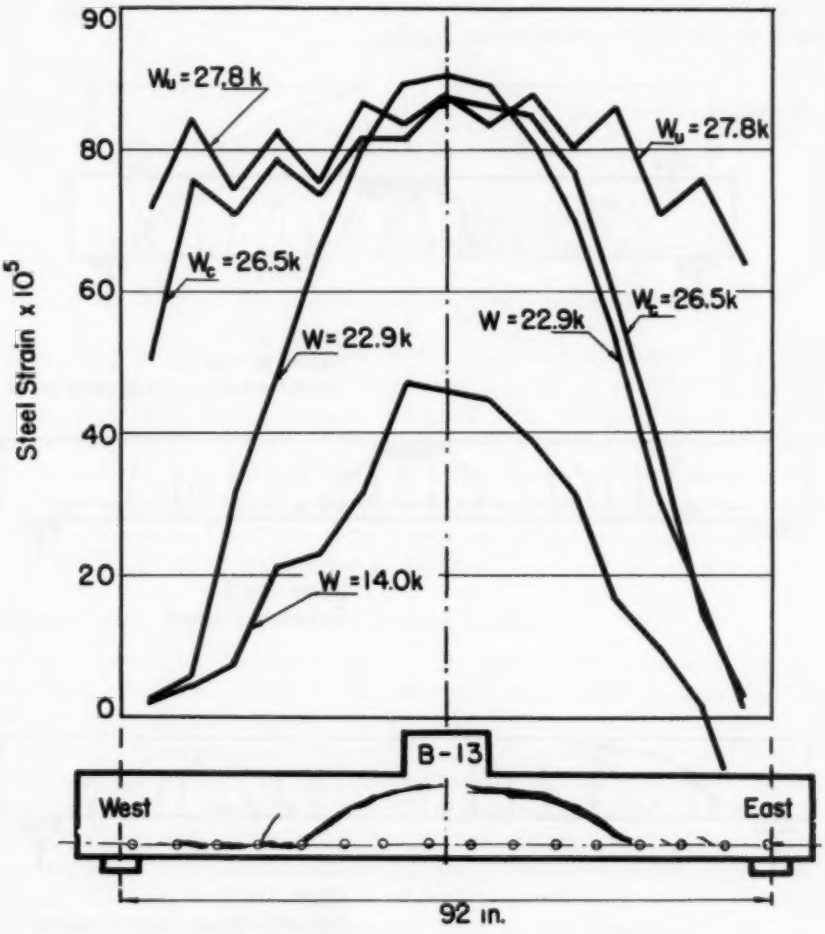

Fig. 5—Steel strain distributions for Beam B-13

and sometimes the inclined crack became horizontal before reaching the region in which the load was applied. For the beams under uniform load, the inclined crack intersected the tensile steel at a point about 0.11 of the span, on the average, from the center line of the support. The inclined portion of the crack intersected middepth always in the neighborhood of 0.15 of the span from the support, and before final collapse always became horizontal for a certain length in the compression zone. Some of the beams developed symmetrical inclined crack patterns about midspan before final collapse. In others, only one major inclined crack formed at one end. Three typical crack patterns are shown for the beams under uniform load in Fig. 6, one for a flexural failure, one for Beam D-4 that failed in flexure after the formation of a major inclined crack, and one for a typical shear-compression failure.

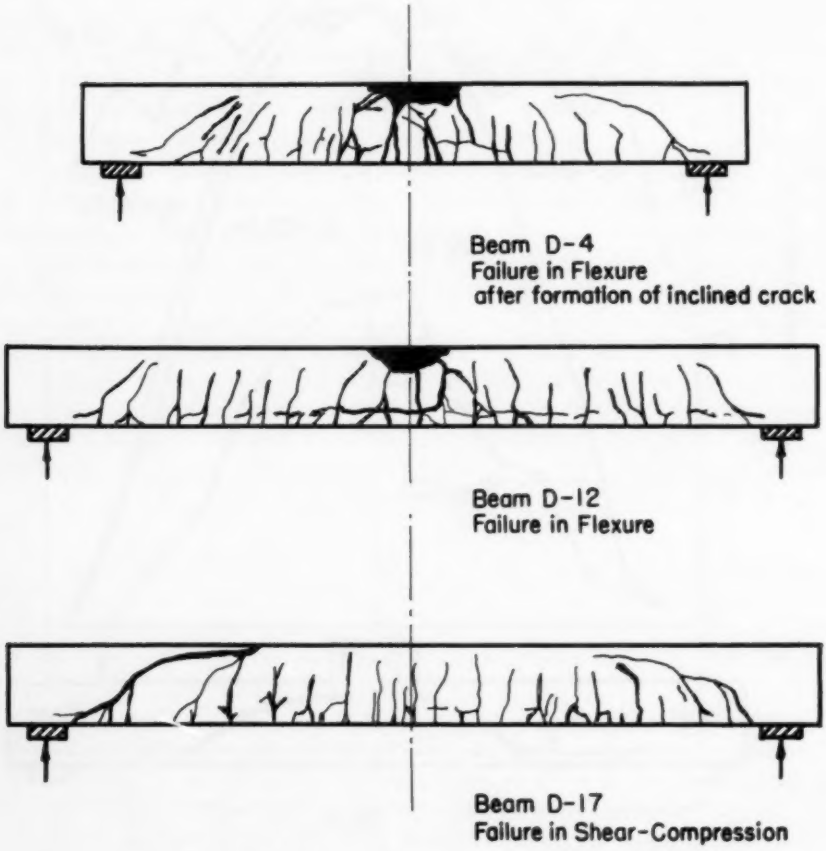

Fig. 6—Crack patterns: Beams under uniform load

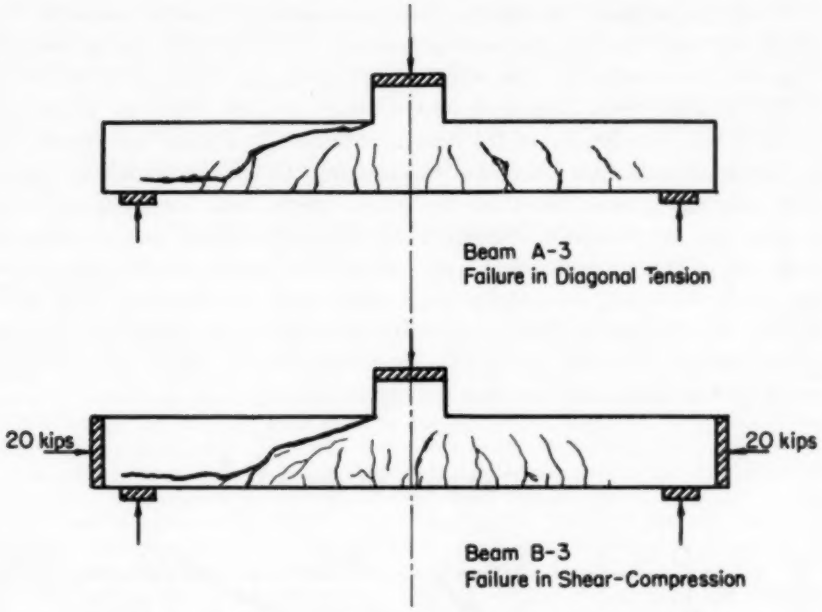

Fig. 7—Crack patterns: Beams with and without axial load

The presence of the axial load in the stub beams affected the crack pattern as follows: flexural cracks started forming at a higher load, they were not as high, and there were fewer of them. Inclined cracks formed closer to midspan section in the beams with axial load, they were not as high, and their development was slower than in the corresponding beams with no axial load. This is illustrated by Fig. 7 which shows crack patterns for Beam A-3 with no axial load, and for its companion specimen B-3, tested under an axial load of 20 kips.

Concrete strain distribution across inclined cracks

The concrete strain distribution for an uncracked section can be assumed to be a straight line with a fair degree of accuracy, especially in the compression zone. However, the appearance of an inclined crack in the compression zone alters this condition. To investigate the distribution of strains and stresses in the neighborhood of an inclined crack, strains were measured in some of the beams under uniform load, at several locations above and below an inclined crack. Since the exact location of the inclined crack could not be predicted in advance, strain gages were placed after the crack had appeared, and the measured strains obtained were, therefore, those at the section after the inclined crack had progressed through it.

In all of the beams in which these measurements were made, it was found that the concrete below the inclined crack carried an appreciable amount of compression. The strain distribution appeared to be made up of two straight lines displaced with respect to each other, as illustrated in Fig. 8 which is typical of the results obtained. The solid line represents the strain distribution when the load causing the inclined crack to appear first was again reached, after the strain gages had been applied. The broken line represents the strain distribution at failure of the beam by crushing of the concrete above the crack. The strain distribution below the crack remained essentially unchanged from the cracking load until failure. The strains at failure were extrapolated from curves of concrete strain versus midspan deflection, and the strains above the inclined crack are probably smaller than the actual strains.

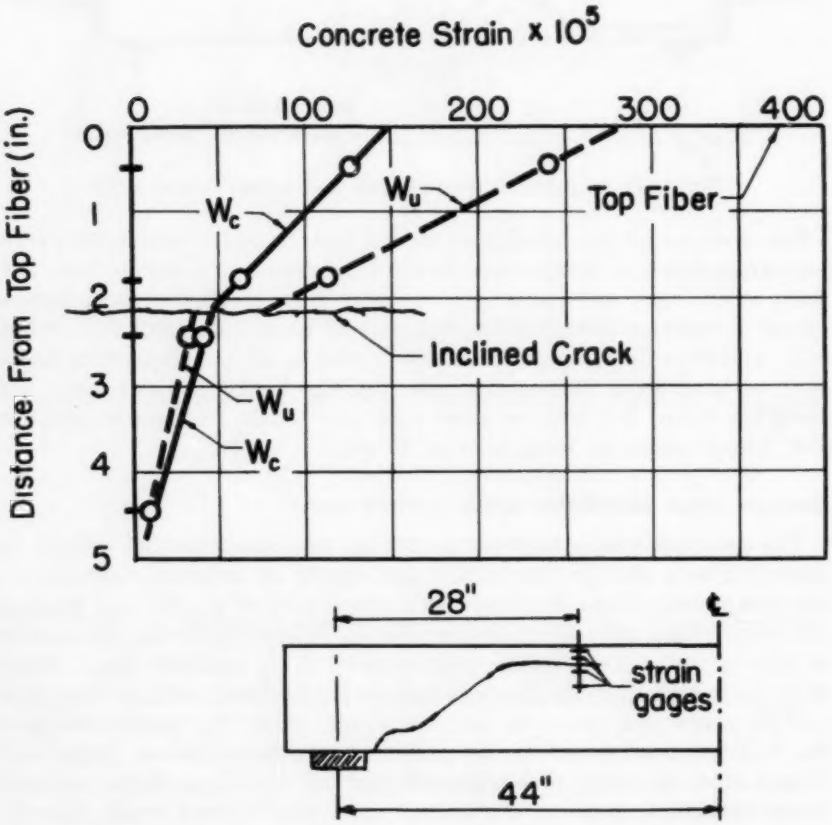

Fig. 8—Concrete strain distribution across an inclined crack

These measurements serve only the qualitative purpose of showing to what extent there were compressive strains below an inclined crack. It is not known, however, how these results are affected by the fact that the measurements were made above and below practically horizontal cracks, or by the fact that the load causing the crack to form was removed before applying the strain gages.

FRAME MEMBERS UNDER UNIFORM LOAD

Test program

The final stage of the investigation consisted of the design and test of 24 frame members designated F-1 through F-24 in the order they were tested. Closely spaced stirrups were placed in the legs up to the bottom fiber of the horizontal member to prevent failures there. Only Frames F-9 through F-11 had web reinforcement in the form of bent bars in the horizontal member. The properties, behavior, and test results for these three frames are discussed in the appendix.

All frames were tested under a uniform load simulated by ten concentrated loads equally spaced along the span. Axial load was applied at the legs by an independently operated hydraulic jack. The ratio of axial to vertical load was kept constant throughout any one test. The general arrangement and details of the reinforcement are shown in Fig. 9. The test setup is shown in Fig. 10.

The following major variables were studied in this series: ratio of axial to vertical load (0.17, 0.26, and 0.33); ratio of maximum positive moment to total moment between column face sections (from 0.2 to 0.8); clear span length (6, 8, and 10 ft); concrete strength (from 2500 to 5700 psi); and steel ratio (from 0.67

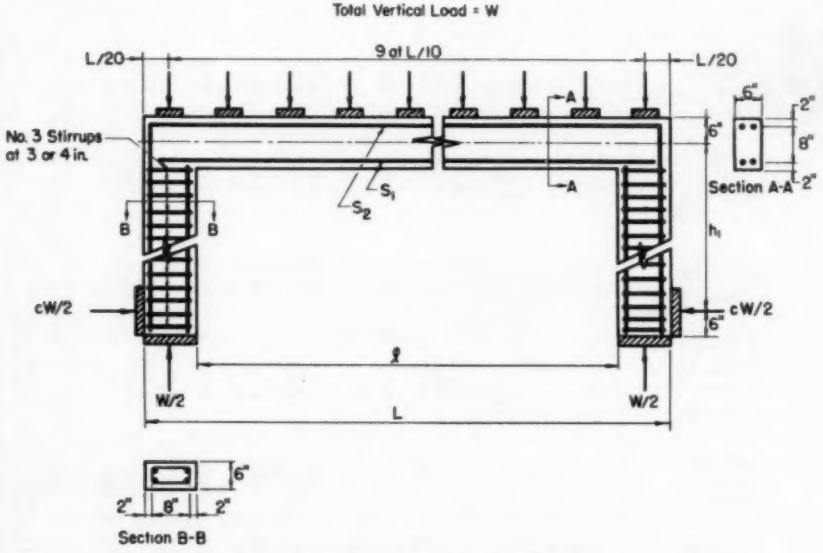

Fig. 9—Details of frames without web reinforcement

TABLE 3—PROPERTIES OF BEAMS

Mark	Clear span length l, in.	Ratio of axial to vertical load, c	Effective leg length h_1, in.†	Moment ratio, M_B	Positive reinforcement			Negative reinforcement			Concrete strength f'_c, psi‡	Modulus of rupture f'_r, psi‡
					Yield point stress f_y, ksi‡	Bars S_1	Steel ratio p, percent	Yield point stress f_y, ksi‡	Bars S_2	Steel ratio p', percent		
For all frames: $b = 6$ in.; $d = 10$ in.; $h = 12$ in.												
F-1	72	0.17	27.0	0.73	48.0	2-#7	2.00	48.0	3-#4	1.00	4100	480
F-2*	72	0.17	27.0	0.67	47.6	2-#7	2.00	47.6	3-#4	1.00	5000	590
F-12	72	0.17	46.0	0.20	49.4	2-#4	0.67	50.6	2-#8	2.63	3000	418
F-15	72	0.17	27.0	0.67	47.3	2-#7	2.00	46.8	3-#4	1.00	2520	375
F-16	72	0.17	27.0	0.67	47.3	3-#5	1.55	48.2	2-#4	0.67	3440	392
F-21	72	0.17	27.0	0.67	46.3	3-#4	1.00	45.2	2-#4	0.67	3990	408
F-5	72	0.33	17.0	0.49	45.8	3-#4	1.00	45 8	3-#4	1.00	3500	400
F-6*	72	0.33	17.0	0.49	52.3	3-#4	1.00	52.3	3-#4	1.00	3400	433
F-13	72	0.33	12.0	0.75	47.5	2-#7	2.00	48.3	2-#4	0.67	3090	416
F-18	96	0.17	33.0	0.68	46.8	2-#7	2.00	46.5	3-#4	1.00	3700	342
F-19	96	0.33	16.5	0.68	46.8	2-#7	2.00	46.1	3-#4	1.00	3040	437
F-3	120	0.17	43.0	0.63	47.9	2-#7	2.00	47.1	3-#4	1.00	4130	430
F-4*	120	0.17	43.0	0.62	42.4	2-#7	2.00	45.6	3-#4	1.00	3730	491
F-22	120	0.17	43.0	0.63	41.8	3-#5	1.55	44.8	2-#4	0.67	3680	368
F-23	120	0.17	43.0	0.63	46.5	3-#4	1.00	46.8	2-#4	0.67	3300	400
F-24	120	0.17	43.0	0.63	46.4	2-#7	2.00	46.1	2-#4	0.67	3710	392
F-17	120	0.26	27.0	0.63	44.9	3-#5	1.55	45.6	2-#4	0.67	3340	387
F-7	120	0.33	27.0	0.48	48.0	3-#4	1.00	44.9	3-#4	1.00	4000	511
F-8*	120	0.33	27.0	0.48	47.9	3-#4	1.00	47.9	3-#4	1.00	5200	550
F-14	120	0.33	15.0	0.80	51.5	2-#8	2.63	49.0	2-#4	0.67	3980	425
F-20	120	0.33	21.5	0.63	46.8	2-#7	2.00	45.9	3-#4	1.00	3830	408

*Frames had the negative steel cut-off at approximately one third of the clear span.
†h_1 = distance from the point of application of the axial load to mid-depth of horizontal member.
‡Average values for the bars used in frame.
‡Average values obtained from control cylinders and beams cast and cured together with the corresponding frame and tested the same day the frame test was performed.

to 2.33 percent). All frame members had a 6 x 12-in. cross section with an effective depth of either positive or negative steel of 10.0 in. The properties of the frames without web reinforcement are given in Table 3. The frames are grouped according to their clear span lengths and, within each span length, in ascending order of ratio of axial to vertical load.

The first eight frames (F-1 through F-8) were designed to have ratios of axial to vertical load and ratios of moment to axial load similar to those encountered in typical reinforced concrete box culverts under deep fills, as designed by the Corps of Engineers. The statics of the frame members tested are such that for any given set of section dimensions, clear span length, vertical to horizontal load ratio, and for one ratio of moment to axial load at any section, there corresponds only one length of vertical member. After the first eight frames were manufactured and tested, all other test specimens were designed with the purpose of isolating or extending the range of the major variables.

Materials, equipment, and testing procedure

Most of the essential features of the frame tests were identical to those for the simply supported beams under uniform load, previously described. Only characteristics not identical to those of simple beams are described in this section.

Reinforcing steel—All reinforcement in the frame members consisted of deformed, intermediate grade bars meeting the requirements of ASTM A 305, with yield point stresses ranging from 41,800 to 52,300 psi, an average maximum strength of 74,000 psi, and elongations at failure of 18 to 25 percent in a gage length of 8.0 in.

Fabrication—All frames were cast in steel forms with adjustable end plates, with the legs extending upward. The negative steel was held in position by two or three chairs made from 1/4 in. mild steel bars. To hold the positive steel in position while casting, it was suspended by wires from transverse steel bars resting on the top of the steel form. The wires and transverse bars were removed from the forms as soon as casting was completed. Two to three batches of concrete were necessary for the frames. For the specimens with long vertical members, all of the third batch was usually placed in the legs, and in such cases the reported compressive strengths and moduli of rupture are the average of the two first batches, corresponding to the horizontal portion of the frame. All the concrete was cured under wet burlap for 5 to 7 days. The frames were then removed from the forms and stored with their control specimens in the laboratory until tested.

Equipment—The legs of the frames rested on 6 x 12 x 1-in. bearing blocks seated with plaster on concrete abutments. The axial loading equipment was completely independent of the vertical loading system. It consisted of a hydraulic jack operating against one end of the frame. with the reaction to the jack supplied by tension rods operating against the other end (Fig. 10). To allow the ends of the legs to rotate, a 4 in. diameter half round rocker welded to a 6 x 12 x 1½-in plate was supplied at one end, and a 1½-in. diameter steel ball was placed between depressions in two 6 x 12 x 1½-in. steel plates at the other end. To distribute the load evenly, ⅛ in. thick leather pads were used between the bearing plates and the sides of the legs.

Measurements—In addition to three dials to record deflections at the quarter points of the clear span of the horizontal member, two other dial indicators were placed at the ends of the legs to record the horizontal movements of the legs. Steel strains were measured in all the frames in both positive and negative rein-

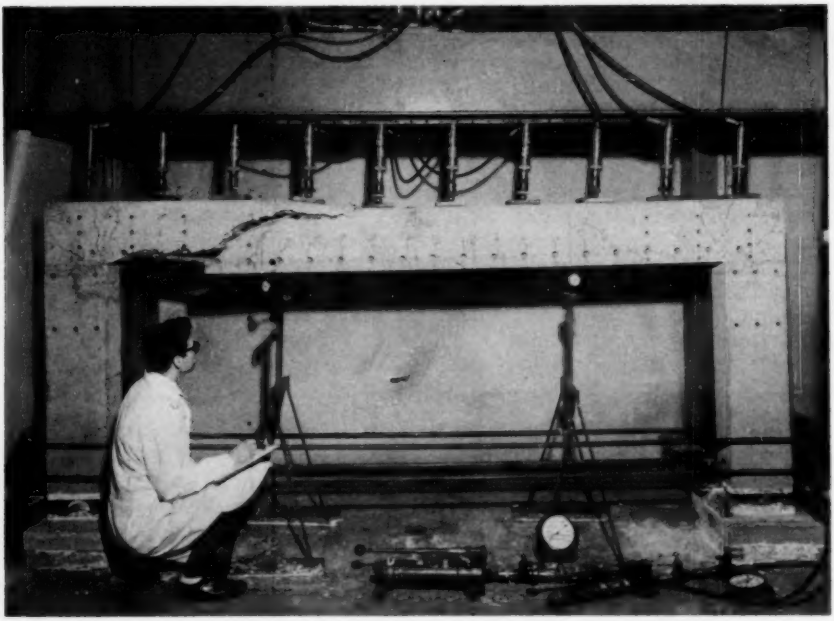

Fig. 10—Test setup showing Frame F-24 after failure

forcement in outside bars, usually at opposite sides of the specimen. The strains were measured over contiguous 6-in. gage lines along the horizontal member. The number of gage lines varied from 18 to 42, depending on the length of the specimen. Concrete strain readings were taken over 10-in. gage lengths, chiefly for the purpose of checking the symmetry of the loading, at both ends of Frames F-2 through F-24. The load was measured by calibrated pressure gages in the hydraulic loading systems.

Testing procedure—Special care was taken when placing the specimens into the testing frame to insure that the axial load was actually applied in the mid-vertical plane, and that the vertical load was actually vertical. A small initial pressure was given to the jacks, and adjustments were then made. Axial and vertical loads were applied in increments by simultaneous operation of the two pumps to keep the ratio of axial to vertical load constant. Load was applied in 8 to 20 increments up to final collapse, the increments being smaller in the last stages of the tests. For each increment, measurements were taken, and cracks were observed and marked with ink. Photographs were taken at critical stages in the test.

TEST RESULTS

Presentation of test data

Test results for the frames without web reinforcement are given in Table 4. Total vertical loads and axial loads are given at both the inclined tension cracking load and at final collapse. Midspan deformations were obtained in much the same manner as those for the simply supported

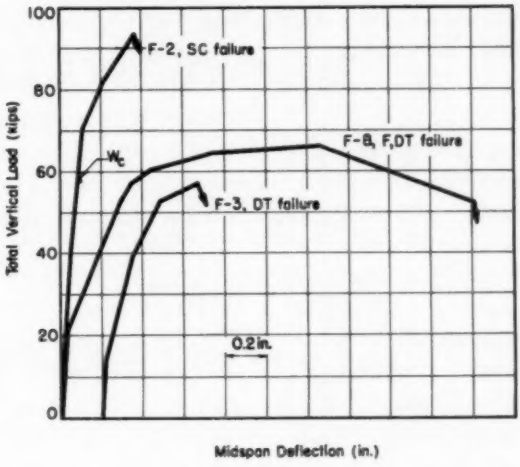

Fig. 11 — Typical load-deflection curves for frames without web reinforcement

beams. The last column of Table 4 lists the modes of failure. The load values include live load only, since the heaviest frame member weighed 1.3 kips. Typical load-deflection curves, crack patterns, and steel strain distributions for positive and negative reinforcement are shown in Fig. 11-15.

Behavior and modes of failure

Before the formation of flexural cracks, the behavior of the frame members was similar to that of simply supported beams under uniform load, allowing for the facts that both positive and negative flexural cracks form in the frame members and that negative cracking tends to be concentrated towards the re-entrant corner of the frames. The same three major modes of failure observed in the simply supported beams were evident in the frame members.

Failure in flexure — Only one of the frames tested failed in this manner, F-23. Failure was accompanied by large deformations (Table 4), after yielding of the positive and negative reinforcement, with little or no inclined cracking. Flexural tension failure occurred in the negative moment region, with extensive crushing of the concrete at the corner (Fig. 12).

Failures in diagonal tension — Five 10-ft frames and one 8-ft frame failed in this manner. All of these failures were sudden and complete, with no previous indication of inclined cracking. All the frames had undergone relatively little deformations at the time of failure.

Combinations of flexure and diagonal tension — Frames F-7 and F-8 failed suddenly at the formation of the first fully developed inclined crack, but

TABLE 4—TEST RESULTS

	Inclined tension cracking				Failure				
Mark	Total vertical load W_c, kips	Axial load N_c, kips	Midspan tensile steel strain $\times 10^5$	Midspan deflection, in.	Total vertical load W_u, kips	Axial load N_u, kips	Midspan tensile steel strain $\times 10^5$	Midspan deflection, in.	Mode of failure†
F-1	61.6	9.3	65	0.150	83.6	12.7	75	0.300	SC
F-2	66.0	11.0	72	0.097	93.5	14.3	95	0.367	SC
F-12	72.0	12.0	152	0.136	91.0	15.2	630+	0.315	SC
F-15	46.2	7.7	52	0.066	77.0	12.9	110	0.428	SC
F-16	54.6	9.1	80	0.096	82.0	13.8	100	0.506	SC
F-21	60.0	9.9	124	0.166	74.5	12.7	143	0.411	SC
F-5	90.3	30.8	122	0.139	96.8	30.5	142	0.202	SC
F-6	88.0	27.4	130	0.134	110.0	34.9	174	0.245	SC
F-13	52.0	17.3	65	0.090	84.5	29.2	88	0.413	SC
F-18	59.0	9.8	110	0.225	59.0	9.8	110	0.225	DT
F-19	54.6	18.2	73	0.184	64.0	21.3	92	0.350*	SC
F-3	57.2	9.5	122*	0.470	57.2	9.5	122	0.470	DT
F-4	51.7	8.8	83*	0.378	51.7	8.8	83	0.378	DT
F-22	46.2	7.7	146*	0.407	46.2	7.7	146	0.407	DT
F-23	—	—	—	—	42.0	7.0	300	1.290	F
F-24	46.2	7.7	131	0.360	48.0	8.3	155	0.409	DT, SC
F-17	50.4	13.4	154	0.390	52.5	14.5	170	0.456	DT, SC
F-7	60.5	20.8	500*	1.195	60.5	20.8	500	1.195	DT, F
F-8	66.0	22.0	559*	2.146	66.0	22.0	559	2.146	DT, F
F-14	51.0	17.2	84+	0.330	51.0	17.2	84+	0.330	DT
F-20	54.6	18.0	128*	0.284	54.6	18.0	128	0.284	DT

*Extrapolated values.
†SC = shear-compression failure; DT = diagonal tension failure; F = flexural tension failure.

after considerable yielding of both positive and negative reinforcement had taken place, and after crushing had begun at the inside corner and at midspan. Practically no additional load was carried beyond that causing yielding of positive and negative reinforcement.

Failures in shear-compression — All of the 6-ft frames and one 8-ft frame failed in shear-compression. Inclined tension cracking began one or two load increments after the flexural cracks stopped developing. The cracks were small at first, and occurred at either side of the point of contraflexure near middepth. The pattern of inclined cracks was sometimes symmetrical about midspan; at other times an inclined tension crack formed first at one end, and later at the other end, or not at all. As load increased, the inclined tension cracks progressed toward midspan, passing above existing flexural cracks. At this point it is desirable to note that, while in most of the simply supported beams under uniform load the inclined crack always formed from an existing flexural crack, the opposite was true for the frames and most of the inclined cracks formed at first independently of the existing flexural cracks. The inclined cracks also extended toward the column face by splitting along the bottom steel. The members collapsed before, after, or simultaneously with yielding of the reinforcement, and crushing of the concrete usually occurred both at the inside corner and at the top of the inclined crack.

Borderline cases between shear-compression and diagonal tension — The members failing in this manner (F-17 and F-24) were able to carry little load beyond that causing the first inclined tension crack to form. In fact, it is believed that if the load had been allowed to remain on the member long enough, final collapse would have occurred without further increase in load.

Crack patterns

Typical crack patterns for the frame members are shown in Fig. 12. Marked in the sketches at one end only is the section at which the bending moment at middepth was zero, the point of contraflexure. With regard to shear failures, two general types of crack patterns were observed in the frame tests.

Failures in the negative moment region — The failure crack extended from the inside corner at the column face towards midspan at an angle ranging from 40 to 60 deg with the horizontal. The heavier crushing was always observed in the compression zone at the column face section. Frames F-12, F-5, F-7, and F-8 fall in this category. For the two last frames mentioned, heavy crushing occurred also at midspan, and the flexural cracks were quite high and wide at failure, since the shear capacity was reached at the same time as the flexural capacity.

Failures in the positive moment region — All of the remaining frame members tested with no web reinforcement failed in this manner. The point at which the crack crossed middepth was always in the positive moment re-

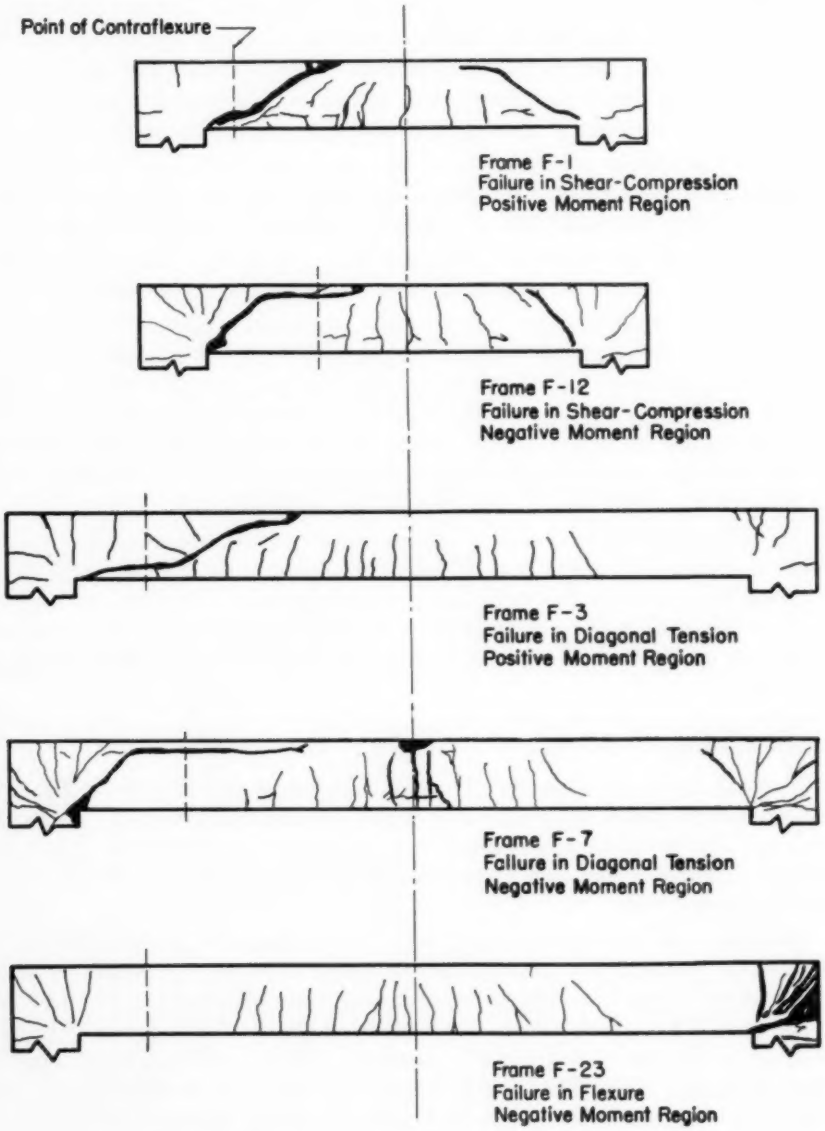

Fig. 12—Crack patterns: Frames without web reinforcement

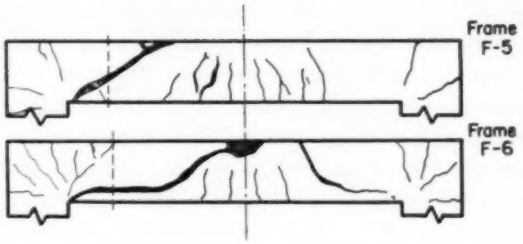

Fig. 13—Crack patterns for Frames F-5 and F-6

gion. The failure crack was inclined 40 to 60 deg to the horizontal until it crossed the positive steel, which it then followed to the inside corner at column face. The failure crack also progressed toward midspan, sometimes becoming horizontal along the top steel. At other times the crack did not become horizontal along the top, and for these cases the crushing was heavier at the top of the inclined crack than at the corner.

The only factor that seemed to determine whether the frames failed at the positive or at the negative moment was the ratio of maximum positive moment at midspan to maximum negative moment at the column face. All of the frames that failed in the negative moment region had a maximum moment at the column face larger than the maximum positive moment at midspan. The ratio of maximum positive to negative moments can best be expressed as the ratio of positive moment at midspan to total moment between column faces, herein denoted as M_R, the moment ratio (for M_R greater than 0.5, the moment at midspan is larger than the moment at the column face).

With reference to Fig. 13, it can be observed that Frames F-5 and F-6 failed quite differently although they were almost identical (F-6 had the negative steel cut off at the one-third points of the clear span, while F-5 had the negative steel continuous). For Frame F-6 the inclined crack causing failure ended at midspan, with heavy crushing there, whereas F-5 failed typically at the negative moment region. The value of the moment ratio for these two frames was 0.49; that is, the maximum positive and negative moments were almost equal. If the ratio of maximum positive to negative moment is the controlling factor for the location of the failure region, this apparent discrepancy can be explained, since a frame with a moment ratio of 0.50 would have equal chances of failing at the positive or negative moment regions. However, this conclusion must be taken with caution because the number of frames that failed at the negative moment region was relatively small.

Steel strain distributions

The redistribution of steel strain at the formation of a fully developed inclined crack, that is at the cracking load, was observed in the short-

span frames in a more marked fashion than in the simply-supported beams. At the critical section, the steel strain changed suddenly from compression to a large tension (Fig. 14). Fig. 15 shows the steel strain distribution for the negative steel of Frame F-19, typical of the results obtained for other frames. The negative steel for the frames failing in the positive moment region was not affected by the presence of an inclined crack. For the frames that failed in the negative moment region, the sudden change in steel strain at the formation of the inclined crack was observed in the negative steel.

STUDY OF RESULTS — SHEAR FAILURES

Effects of variables

Cracking load is defined in this paper as the load at the formation of a fully developed inclined crack. For short and deep beam members, in which the development of inclined cracks is slow, a crack was considered to be fully developed when it caused a marked change in steel strains. For medium long members, the formation of a fully developed inclined crack is a well defined phenomenon; the crack develops initially to a considerable length, easily detectable by visual observation. In this case, the value of the cracking load obtained visually agrees well with that obtained from steel strains. Finally, for long slender members, the form-

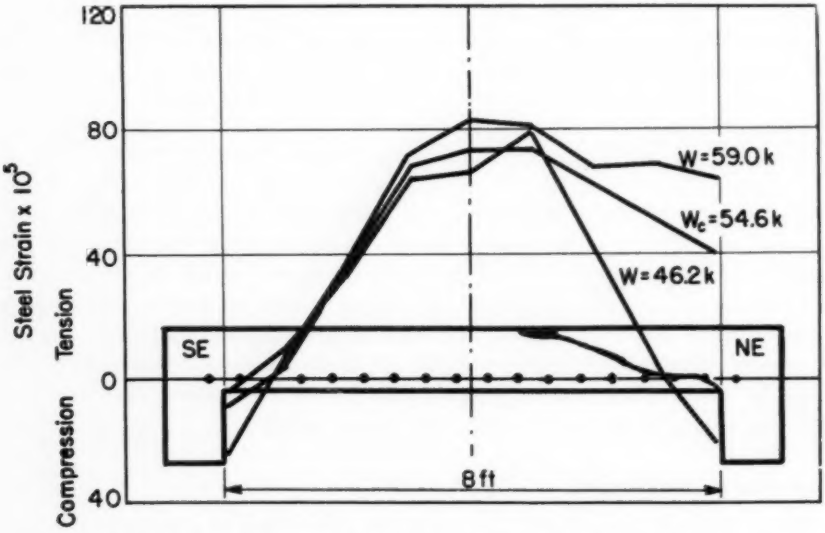

Fig. 14—Positive steel strain distributions for Frame F-19

ation of the first fully developed inclined crack causes complete collapse of the specimen, and, by definition, cracking and failure loads are identical.

From the above discussion, the effect of what could be called "slenderness" of the member is most apparent, whether it be the ratio of span to depth, or the ratio of shear span to effective depth: all other conditions being the same, the shorter a beam member, the larger the ratio of the ultimate shear capacity to the cracking load. Also, the longer the member, the smaller the cracking load or the ultimate shear capacity (Tables 2 and 4).

It was found that, all other conditions being the same and within the ranges studied, the capacity of the members at both cracking and final collapse is proportional to the square root of concrete strength, as measured from 6 x 12-in. cylinders (comparing, for example, Beams D-9, D-10, and D-11, or Frames F-1, F-2, and F-15, in which concrete strength was the only variable). This variation, for the cracking load, is in agreement with experimental evidence obtained in other investigations.[2]

A study of the test results in Table 3 and 4 reveals also that shear capacity is roughly a linear function of tensile steel percentage. Varia-

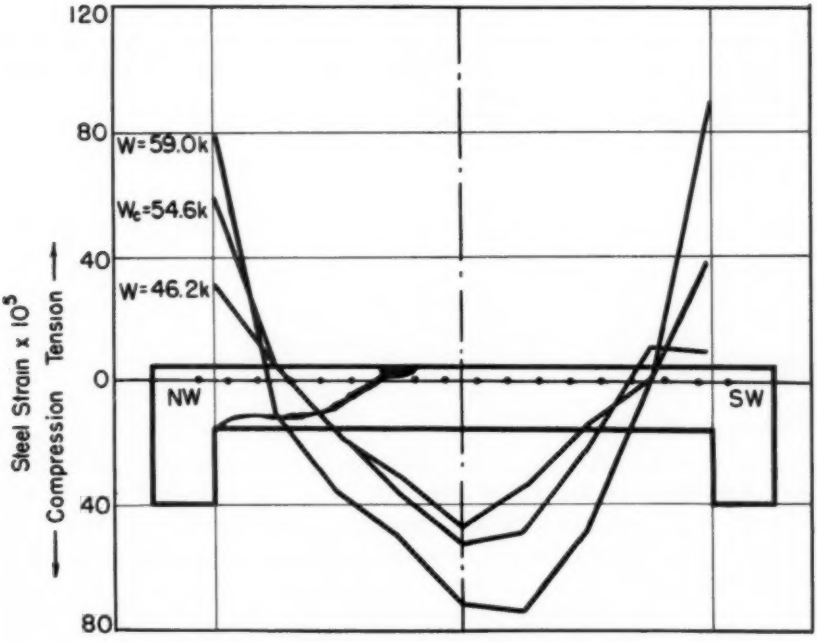

Fig. 15—Negative steel strain distributions for Frame F-19

tions in the negative steel percentage in the frames failing at the positive moment region were found to have little influence on the strength of the members.

The type of vertical loading has a profound influence on the shear carrying capacity of a member. Members with almost identical properties failed under uniform load at loads much larger than under concentrated loads. For those beams in which comparisons could be made (e.g., D-1 and A-14), allowing for differences in concrete strength and small differences in span length, the ratio of the capacity under uniform load to the capacity under symmetrical concentrated loads was always in the neighborhood of 1.5. For a simply supported beam failing in flexure, the ratio is 2.0. This again suggests that shear strength is to a certain degree dependent on moment, or in other words, that there is an interaction between moment and shear in a member which affects its shear capacity.

The presence of axial load had a relatively small influence on the shear capacity of the members tested, and this influence was somewhat larger for the ultimate shear capacity than for the cracking load capacity. Comparing the simply supported beams tested with and without axial load, the effect of the axial load can be summarized as follows:

For the beams with the longest spans, which failed in diagonal tension both with and without axial load, the cracking loads were not significantly different.

For the beams with the shortest spans, which failed in shear compression both with and without axial load, both the cracking load and the failure load were increased appreciably.

In the middle range of span lengths, a beam that failed in diagonal tension without axial load, failed in shear-compression when axial load was present, and failed at a somewhat higher load.

For frames there was a slight but consistent effect of axial load; this effect can be observed by comparing Frames F-18 and F-19 in which the ratio of axial to vertical load was the main variable, and making proper allowance for the difference in concrete strength.

Size and shape of test specimen were not variables in this series of tests, although they probably have some influence on shear capacity and behavior.

Cracking load capacity

Most American investigators now seem to agree that the cracking load should be taken as a measure of the useful shear capacity of a reinforced concrete beam member. Several reasons justify this:

(a) Many members, especially those having the longer spans, fail at the cracking load.

(b) Although the shorter members are able to carry load beyond the formation of the first inclined tension crack, the degree of damage to the members, as judged by their appearance and the nature and width of the

cracks is considerable. Moreover, after fully developed inclined cracks have formed, the member behaves essentially as a tied arch rather than as a beam over a considerable portion of its span (see Fig. 5), and the conventional concepts of flexural action are no longer applicable. This is especially important when the members are part of an indeterminate frame.

(c) The use of ultimate, rather than cracking load, as a criterion would require the use of different expressions for predicting the shear strength of members failing in shear compression and in diagonal tension. In view of the uncertainties that still exist regarding the mechanism of shear failure, this refinement is unwarranted.

The test results were therefore correlated in terms of the cracking load. Although several more or less empirical equations were found that could predict the cracking load capacity of the members tested in this investigation with good accuracy, nevertheless, it was considered desirable to correlate the test results by an equation of the general form suggested by Viest.[3] The form chosen has the advantage that it offers a unified approach for members under both concentrated and uniform loads. The equation can be written as follows:

in which
$$\frac{V_x}{bd} = \left[A\sqrt{f_c'} + Bp\left(\frac{Vd}{M}\right)_x \right] D \qquad (1)$$

V_x = shear at a particular location, lb
b = width of member, in.
d = effective depth of tensile reinforcement, in.
f_c' = concrete strength, as determined from 6 x 12-in. cylinders, psi
p = A_s/bd = tensile steel ratio
$\left(\frac{Vd}{M}\right)_x$ = ratio of shear to moment multiplied by the effective depth, computed at the location where V_x is computed, always positive
A, B = dimensional constants
D = a function of the axial load in the member

It must be noted that the expression proposed by Viest[3] did not contain a term for the axial load. He also expressed the equation in terms of the resistance of concrete to principal tensile stresses, subsequently made a function of the square root of concrete strength. From the test data reported in this paper, the following values for the quantities A, B, and D, in Eq. (1) were obtained:

A = 2.14 $lb^{1/2}$ $in.^{-1}$
B = 4600 psi
$D = 1 + 0.04\left(\frac{N}{V}\right)_x$ in which $\left(\frac{N}{V}\right)_x$ is the ratio of axial load to shear at the section where V_x is computed

Eq. (1) expresses the cracking shear V_x as a function of the properties of the concrete and the cross section, and as a function of the ratio of shear to moment at a given location along the span. For simple spans under symmetrical concentrated loads, the applied shear is constant over

the length of the shear span, whereas the cracking shear V_x from Eq. (1) varies from a minimum at the load point to a maximum at the support. Thus, in this case the critical section would occur at the load point where V_x is minimum. Viest[3] has suggested, however, that the critical section not be taken closer than a distance d from the load point or from the support. In other cases,[2] it has been proposed that the critical section be taken at the middle of the shear span. In this investigation, the best correlation between the predictions of Eq. (1) and the results of the tests on simply supported beams with concentrated loads was obtained by taking the critical section at the middle of the shear span, which usually corresponded fairly well to the location of the inclined crack. However, the correlation would not be greatly affected by taking the critical section at a distance d from the support, since the variation of V_x along the span is not great.

For beams and frames under uniform loads, both the applied shear and the cracking shear V_x from Eq. (1) vary along the span length. Consequently, the location of the critical section must be determined in each case either graphically by plotting V_x along the span, or algebraically by equating the derivative of Eq. (1) with respect to x to the slope of the actual shear diagram to determine the point of tangency. For all the simply supported beams and for the frames failing in the positive moment region (except F-6), this procedure was used successfully to predict the cracking loads and also gave a reasonably good indication of the location of the inclined crack. However, for these tests, it was found that substantially the same degree of accuracy could be obtained by taking the critical section at a distance of 0.15 l' toward midspan from the point of zero moment. In general this location corresponded to the average position of the point where the inclined crack crossed middepth of the member.

The application of Eq. (1) to the negative moment region of the frames requires the placing of some limit on the distance of the critical section from the face of the support, since V_x from Eq. (1) varies from a minimum at the column face to infinity at the point of contraflexure. After some study of the limited data available from these tests, it was decided to take the critical section at a distance d from the face of the column. On this basis, Eq. (1) was found capable of predicting the location of the inclined crack in the negative moment region correctly for Frames F-7, F-8, and F-12. For Frames F-5 and F-6 this procedure indicated an almost equal likelihood of the crack being in the positive or in the negative moment region, which is consistent with the observed location of the inclined crack in the positive moment region for Frame F-6 and in the negative moment region for Frame F-5.

The cracking load was predicted by Eq. (1) with satisfactory accuracy for the beams under concentrated loads, for the simply supported beams under uniform load, and for the frames failing in the positive moment region, as can be seen in Table 5 and Fig. 16. However Eq. (1) was found to give extremely conservative values of cracking load for the four frames (F-5, F-7, F-8, and F-12) failing in the negative moment region, and for Frame F-6 which failed in the positive moment region in a rather

TABLE 5—COMPARISON OF MEASURED AND COMPUTED VALUES AT CRACKING LOAD

Mark	At critical section V/bd, psi		Ratio measured computed
	Measured	Computed	
Beams under two-point loads and retests*			
L-1	251	272	0.92
L-2	218	222	0.98
L-2a	241	257	0.94
L-3	201	213	0.94
L-4	193	192	1.00
L-5	191	187	1.02
		Average	0.97
L-2R	230	222	1.04
L-2aR	266	257	1.08
L-3R	233	213	1.09
		Average	1.06
Frames: $M_R > 0.5$†			
F-1	221	219	1.01
F-2	232	237	0.98
F-15	153	182	0.84
F-16	192	195	0.98
F-21	222	189	1.17
F-13	181	197	0.92
F-18	221	197	1.12
F-19	202	186	1.09
F-3	224	201	1.11
F-4	201	191	1.05
F-22	185	181	1.02
F-24	177	191	0.93
F-17	202	172	1.17
F-14	211	207	1.02
F-20	206	193	1.07
		Average	1.03
		Standard deviation	0.090
Frames: $M_R < 0.5$†			
F-12	326	274	1.19
F-5	270	190	1.42
F-6	268	192	1.40
F-7	355	199	1.78
F-8	382	220	1.74
		Average	1.51

Mark	At critical section V/bd, psi		Ratio measured computed
	Measured	Computed	
Stub beams*			
A-1	172	182	0.94
A-2	165	175	0.94
A-3	128	137	0.93
A-4	132	152	0.87
B-1	248	190	1.30
B-2	195	180	1.08
B-3	172	167	1.03
B-4	157	168	0.93
A-11	236	290	0.81
A-12	220	235	0.94
A-13	177	198	0.89
A-14	205	196	1.04
A-15	184	180	1.02
B-11	315	294	1.07
B-12	247	249	0.99
B-13	220	225	0.98
B-14	198	212	0.93
B-15	177	201	0.88
		Average	0.98
		Standard deviation	0.101
Beams under uniform load†			
D-15	183	174	1.05
D-14	194	188	1.03
D-16	203	209	0.97
D-13	151	149	1.01
D-17	168	206	0.82
D-5	193	192	1.00
D-4	241	221	1.09
D-9	186	191	0.97
D-10	211	210	1.00
D-11	279	255	1.09
D-7	183	180	1.02
D-6	203	198	1.02
D-1	225	219	1.03
D-2	226	239	0.94
D-8	236	256	0.92
D-3	234	218	1.07
D-18	188	200	0.94
		Average	1.00
		Standard deviation	0.065

*Critical section at $a/2$.
†Critical section at point where shear line is tangent to resisting curve.
‡Critical section at a distance d from column face.

unusual manner. It should be noted that Frames F-5 and F-6 were almost identical but failed in markedly different manners as can be seen in Fig. 13.

The five frames for which Eq. (1) gives cracking loads much lower than those actually obtained have one characteristic in common. In all cases the negative moment at the column face was greater than the positive moment at midspan ($M_R < 0.5$), and in all but one of these frames the failure occurred in the negative moment region. This suggests a phenomenon of higher resistance to inclined cracking in the region of negative moment or in a region close to a column, but no conclusion of this nature can be justified in view of the relatively few failures of this type that were obtained in these tests.

In summary, the cracking load can be predicted by an equation in which the numerical constants and the locations of the critical sections for the different types of members were determined from the test results reported in this paper. For members failing in the positive moment region (simply supported beams, and frames in which M_R is larger than 0.5), the results are much more reliable than for members failing in the negative moment region, since only five frames were designed with moment ratios smaller than 0.5

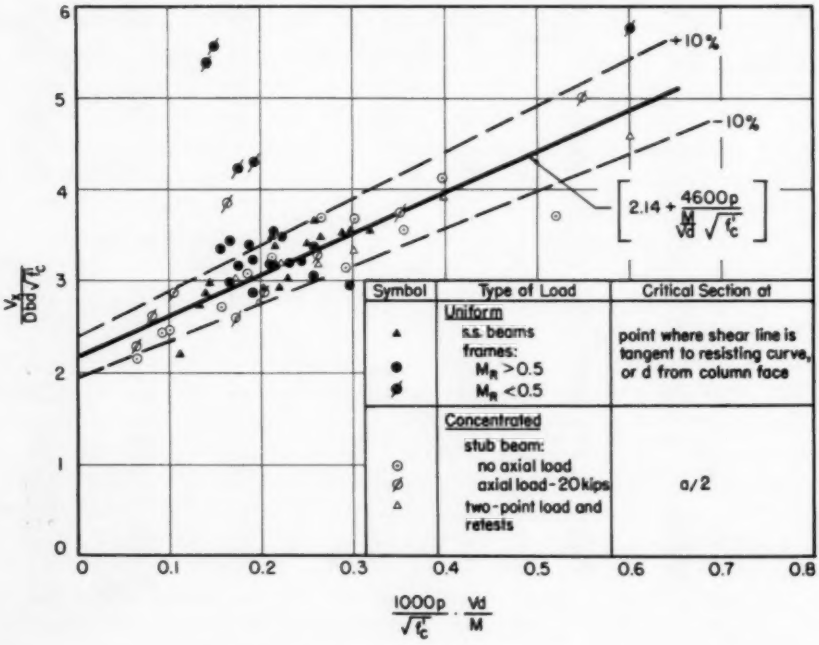

Fig. 16—Cracking load capacity

Capacity beyond cracking load

It was not found possible to develop reliable and general methods for predicting the capacity of the members beyond the cracking load, although, for isolated series of tests, the capacity beyond cracking could be expressed in terms of the known variables. For example, the ultimate strength in shear of the beams under uniform load could be predicted by an expression of the type developed by Laupa, *et al.*,[4] multiplied by a constant factor. However, no correlation was found when this expression was applied to the frame members, or to the beams under axial load.

Although some evidence exists regarding the influence of some of the variables on the capacity beyond the cracking load, this influence is not clearly shown in most of the tests because of the presence of other variables.

Two of the major variables, "slenderness" and axial load, were observed to have a definite effect on the capacity beyond cracking. To illustrate their effects in a qualitative manner, the ratio of ultimate to cracking load W_u/W_c, has been plotted in Fig. 17 versus the ratio of a/d for members under concentrated loads, and versus the ratio l'/d for members under uniform load failing in the positive moment region.

Fairly definite transition zones between diagonal tension and shear-compression failures can be observed in Fig. 17. For members under symmetrical concentrated loads, the value of a/d that divides shear-compression failures from diagonal tension failures seems to be about 3.0.

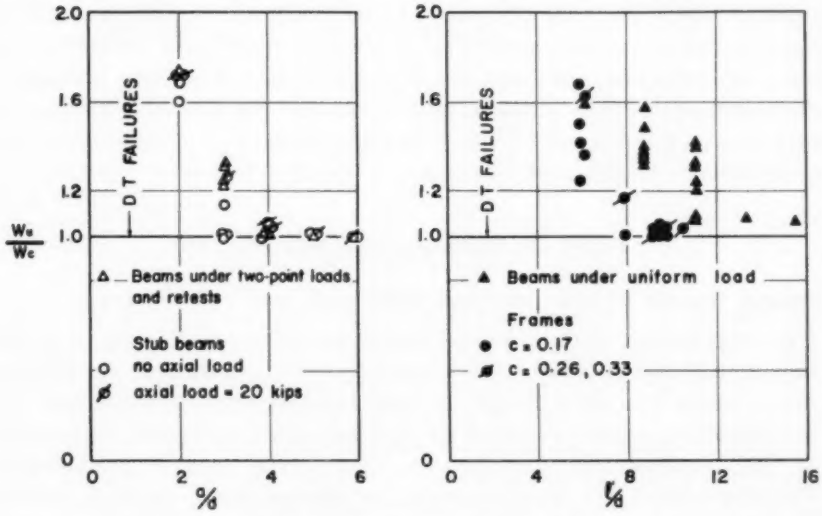

Fig. 17—Capacity beyond cracking load

TABLE 6—MEASURED AND COMPUTED FLEXURAL CAPACITIES

Mark	q	Nd/M	Values of M_f/bd^2f_c'		Ratio measured / computed
			Measured	Computed	
		Beams			
L-6	0.348	0	0.281	0.275	1.02
A-5	0.151	0	0.165	0.137	1.20
B-5	0.163	*	0.183	0.175	1.05
D-12	0.163	0	0.182	0.147	1.24
D-4	0.195	0	0.194	0.172	1.13
		Frames			
F-23	0.095	0.357	0.152	0.141	1.08
F-7	0.112	0.513	0.151	0.141	1.06
F-8	0.092	0.513	0.129	0.117	1.10
				Average	1.11

*Axial load constant = 20 kips.

For members under uniform load, the transition between shear-compression and diagonal tension failures appears to be at a value of $l'/d = 10$ for the frames, and at a value of $l'/d = 12$ or 13 for the simply supported beams. Part but not all of this difference in the transition zones for uniformly loaded members can be attributed to the different conditions of support at the points of contraflexure in the frames, and at the supports in the simple span beams.

It can also be observed in Fig. 17 that the beams or frames with relatively high axial loads (open or solid circles with a diagonal line) had the higher values of the ratio W_u/W_c for a given series of tests, and for the same value of a/d or l'/d.

In summary, both the presence of axial load and the slenderness had some influence on the capacity of a member beyond the cracking load, but this influence could not be determined quantitatively because of the presence of other variables and because of the lack of experimental evidence in which axial load and slenderness varied systematically over a sufficiently large range.

STUDY OF RESULTS — FLEXURAL FAILURES

Flexural capacity of beam members under axial load and bending

A simplified procedure for computing the flexural capacity of underreinforced beam members under axial load and bending is presented in this section. The term "beam" is used here to emphasize the fact that the members under consideration are subjected primarily to moment, as distinguished from columns in which axial load is predominant. Therefore, all of the usual assumptions regarding the flexural strength of beams can be applied. The steel is assumed to be at the yield point

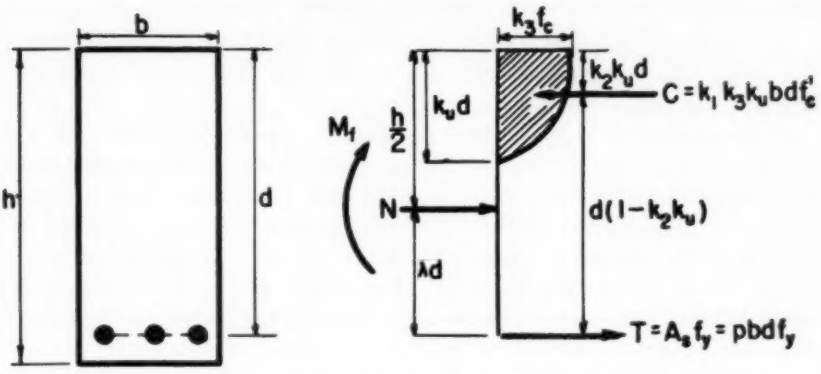

Fig. 18—External and internal forces at ultimate flexural capacity

stress, and to have an indefinitely long flat yield portion. In addition, the contribution of the compressive reinforcement to the flexural strength is neglected because it is rather small, and a margin of safety is provided by not taking this contribution into account.

For the assumptions and simplifications outlined above, Fig. 18 represents the conditions in a section at ultimate flexural capacity. It shows the applied ultimate flexural moment, M_f; the axial load at middepth, N; and the magnitude and location of the compression in the concrete, C, and the tension in the steel, T. By consideration of forces and moments about the tensile steel, denoting $q = pf_y/f_c'$, and after rearranging, the folowing equation can be written:

$$\frac{M_f}{bd^2f_c'} = \frac{M_o}{bd^2f_c'} + \left(1 - \lambda - 2q\frac{k_2}{k_1k_3}\right)(N/bdf_c') - \frac{k_2}{k_1k_3}(N/bdf_c')^2 \quad \ldots(2)$$

in which M_o denotes the resisting moment of the section when no axial load is present

$$\frac{M_o}{bd^2f_c'} = q\left(1 - \frac{k_2}{k_1k_3}\,q\right)$$

For any given section, the ratio of axial load to moment is known. Also known are the properties of the section and the value of λ. Therefore, by solving Eq. (2) for either M_f or N, the load at ultimate flexural capacity can be obtained. The most advantageous way of solving Eq. (2) is by interaction diagrams, since analytical solutions are limited to particular cases and are time-consuming. ACI-ASCE Committee 327 recommends[5] a value $k_2/k_1k_3 = 0.59$ for beams under bending alone. Throughout this paper this value is taken as 0.6.

Comparison of measured and computed flexural capacities

The ultimate flexural capacities for the members failing in flexure was computed using a value $k_2/k_1k_3 = 0.6$. For both beams under axial load and frames, $\lambda = 0.4$.

A comparison of measured and computed values in terms of the dimensionless parameter M_f/bd^2f_c' is made in Table 6. The measured values correspond to the reported ultimate loads, and were obtained at the sections of maximum moment where the failures occurred. It can be seen that all of the computed values are on the safe side, owing to the simplifying and safe assumptions made in deriving Eq. (2). The value $k_2/k_1k_3 = 0.6$ is believed to be conservative. The average value of the ratio of measured to computed values is 1.11.

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A. Feldman, research associate in civil engineering, supervised and reported the tests of beams under two-point loads,[8] and made valuable suggestions throughout the course of the investigation. M. J. Baron, and S. R. Bernaert, formerly research assistants in civil engineering, carried out the tests of stub beams and beams under uniform load, respectively. These tests were first reported in their MS theses.[7,8] G. W. Armstrong, formerly research assistant in civil engineering, assisted in the testing of the frame members. The tests of frames were first reported in full in the doctoral dissertation of the senior author.[9]

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APPENDIX

FRAMES WITH WEB REINFORCEMENT

Introductory remarks

To determine whether a small amount of web reinforcement, in the form of bent bars, would have an appreciable effect on the behavior and strength in shear of reinforced concrete frames, the specimens designated F-9 through F-11 were designed and tested.

After studying several possible arrangements of bent bars and taking into consideration the limited width of the specimens, it was decided to provide a single bent bar at each end of the specimens, symmetrically placed about midspan, and inclined at 45 deg. Frames F-9, F-10, and F-11 were similar to Frames F-6, F-2, and F-4, respectively, except for the presence of bent bars, small differences in steel ratios, and unintentional differences in concrete strength. Comparisons could be made in this way between frames with and without web reinforcement. In all three frames with web reinforcement, one #4 bar of the negative steel was bent down at 45 deg and carried through, symmetrically about midspan. The properties of the frames with web reinforcement are given in Table A-1. The distance a_i is measured from the column face to the point where the inclined bar crossed middepth. This distance was so chosen that the inclined bar would cross middepth at about the location where the failure crack crossed middepth in the companion specimens without web reinforcement. Also listed in Table A-1 for comparison are the properties of the companion frames without web reinforcement.

These frames were cast, cured, and tested in the same manner as their companion specimens without web reinforcement. The measurements taken were also the same. No attempt was made to measure the strain in the inclined bars.

Test results, behavior, and modes of failure

Test results, midspan deformations, and modes of failure for the frames with web reinforcement are given in Table A-2. For convenience the corresponding values for the companion frames without web reinforcement are also shown. Fig. A-1 shows the crack patterns for these three frames, and Fig. A-2 the load-de-

TABLE A-1—PROPERTIES OF FRAMES WITH WEB REINFORCEMENT AND OF COMPANION SPECIMENS WITHOUT WEB REINFORCEMENT

For all frames $b = 6$ in.; $d = 10$ in.; $h = 12$ in.

Mark	F-9	F-6**	F-10	F-2**	F-11	F-4**
Clear span length l, in.	72	72	72	72	120	120
Ratio of axial to vertical load, c	0.33	0.33	0.17	0.17	0.17	0.17
Effective leg length, h_1, in.	17	17	27	27	43	43
Moment ratio, M_B	0.49	0.49	0.67	0.67	0.63	0.62
Positive Reinforcement						
Bars S_1	3-#4	3-#4	2-#7 1-#4	2-#7	2-#7 1-#4	2-#7
Yield point stress,* f_y, ksi	49.7	52.3	47.8	47.6	45.8	42.4
Steel ratio, p, percent	1.00	1.00	2.33	2.00	2.33	2.00
Negative reinforcement						
Bars S_2†	3-#4	3-#4	3-#4	3-#4	3-#4	3-#4
Yield point stress,* f_y', ksi	49.7	52.3	48.4	47.6	47.2	45.6
Steel ratio, p, percent	1.00	1.00	1.00	1.00	1.00	1.00
Web reinforcement						
Bent bar at 45 deg	1-#4	—	1-#4	—	1-#4	—
Yield point stress, ksi	49.7	—	48.4	—	47.2	—
Distance a_1, in.‡	22	—	22	—	24	—
Concrete strength, f_c', psi§	5700	3400	4090	5000	5000	3730
Modulus of rupture, f_r', psi§	408	433	458	590	421	491

*Average values for the bars used in each frame.
†Bars were cut off at approximately one-third of the clear span.
‡a_1 = distance from column face to the point where the 45-deg bar intersected middepth.
§Average values obtained from control cylinders and beams cast and cured together with the corresponding frames and tested the same day the frame test was performed.
**Companion frames without web reinforcement.

TABLE A-2—COMPARISON OF FRAMES WITH AND WITHOUT WEB REINFORCEMENT

Mark	F-9	F-6†	F-10	F-2†	F-11	F-4†
Inclined tension cracking						
Vertical load, W_c, kips	136.5§	88.0	66.0	66.0	57.2	51.7
Axial load, N_c, kips	45.5	27.4	11.0	11.0	9.5	8.8
Midspan steel strain $\times 10^6$	338*	130	60	72	99	83*
Midspan deflection, in.	0.244	0.134	0.110	0.097	0.336	0.378
Failure						
Vertical load, W_u, kips	136.5	110.0	136.4	93.5	70.4	51.7
Axial load, N_u, kips	45.5	34.9	22.8	14.3	11.7	8.8
Midspan steel strain $\times 10^6$	338	174	162	95	154	83
Midspan deflection, in.	0.244	0.245	0.590	0.367	0.548	0.378
Computed flexural failure						
Vertical load, W_f, kips	142.5	121.2	135.0	127.8	70.2	59.4
Ratio, W_u/W_c	1.00	1.25	2.07	1.42	1.23	1.00
Ratio, W_u/W_f	0.96	0.91	1.01	0.76	1.00	0.87
Mode of failure‡	DT	SC	SC	SC	SC	DT

*Extrapolated values.
†Companion frames without web reinforcement.
‡SC = shear-compression failure; DT = diagonal tension failure.
§A small inclined crack formed across the bent bar at a load of about 88 kips. The load causing this crack was not called the cracking load because the inclined crack was not sufficiently developed.

flection curves. The broken lines in Fig. A-2 represent the load-deflection curves for the frames without web reinforcement. When a curve shows a drop in load, it corresponds to the cracking load.

The behavior under load of frames with and without web reinforcement was similar up to a load corresponding approximately to the cracking load of the companion specimen without web reinforcement. From this load onward, two different types of behavior can be distinguished:

(a) A small inclined crack started forming at about the same location as if no web reinforcement were present. However, the presence of an inclined bar at that location prevented the crack from developing further. Additional inclined cracks then developed at sections away from the bent bar, closer to the column face, and final collapse occurred through one of these cracks at a higher load than that of the companion frame without web reinforcement.

(b) An inclined crack formed at the same location as if no web reinforcement were present, as in (a). However, this crack continued to develop under additional load and was the eventual cause of failure which, nevertheless, occurred at a load considerably higher than that of the companion specimen without web reinforcement. The concrete in the vicinity of the crack was heavily damaged.

These two types of crack formation can be seen clearly in Fig. A-1. Frames F-9 and F-11 failed in the manner described in (a), while Frame F-10 failed as described in (b).

Frame F-9 failed in diagonal tension after yielding of the reinforcement at midspan. The initial inclined crack crossing the bent bar was not judged to be fully developed, and there were no sudden changes in the steel strains. Therefore, to be consistent with previous definitions, the failure was classed as diagonal tension, even though it is realized that the classification of shear failures into

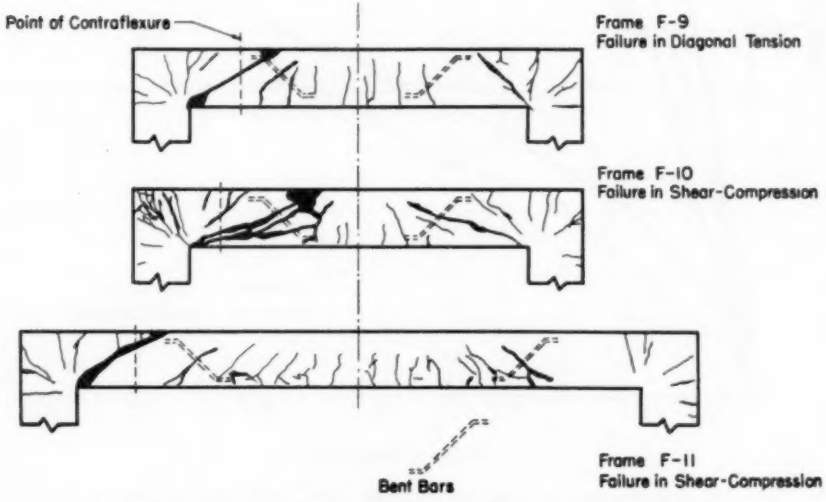

Fig. A-1—Crack patterns: Frames with web reinforcement

shear-compression and diagonal tension has little meaning for members with web reinforcement.

Frame F-10 failed in shear-compression after the formation of a fully developed inclined crack that caused a redistribution of steel strains. Failure was simultaneous with first yielding of both the positive and negative reinforcement.

Comparison with frames without web reinforcement

Listed in Table A-2 are the measured loads and the computed flexural capacities for the frames with web reinforcement and for their companion specimens. Also listed are the ratios of ultimate to cracking load, and of measured shear capacity to computed flexural capacity.

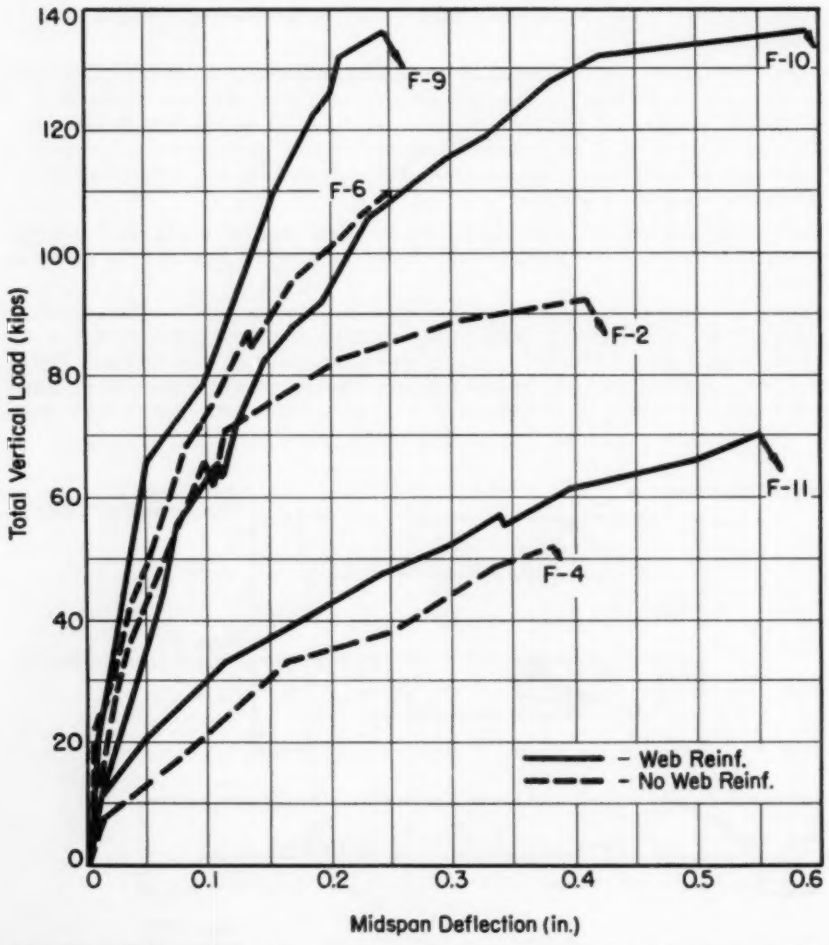

Fig. A-2—Load-deflection curves for frames with web reinforcement and for their companion specimens

For all the frames with web reinforcement the initial inclined tension crack developed at about the same load as for their compainon specimens without web reinforcement. The small differences for Frames F-11 and F-4 can be attributed to differences in concrete strength and steel percentage. For Frame F-9 an initial crack formed, crossing the bent bar, but it was not developed enough to call the load causing it the cracking load. For frames F-10 and F-2 the cracking loads were identical.

The frames with web reinforcement failed practically at their flexural capacities ($W_u/W_f = 0.96$, 1.01, and 1.00, respectively, for F-9, F-10, and F-11). These failures are classed as shear alone because no large deflections took place. The companion specimens failed at loads below their flexural capacities ($W_u/W_f =$ 0.91, 0.66, and 0.87, respectively, for F-6, F-4, and F-2). Thus the presence of web reinforcement increased the capacities of the members, but these increases were not sufficient to prevent the shear failures from occurring.

The influence of the web reinforcement on the load-deflection characteristics of the frames with web reinforcement was negligible up to a load corresponding approximately to the cracking load of the companion specimens with web reinforcement (Fig. A-2). The differences can be attributed to differences in concrete strength and steel percentage. However, above the cracking load, the load-deflection curves for the frames with web reinforcement are considerably steeper than those of their companion specimens.

In summary, the presence of web reinforcement in the form of one symmetrically placed bent bar at roughly the critical section had a small effect on the shear strength and behavior of frames up to a load corresponding to the cracking load of the same frames without web reinforcement. The behavior after the cracking load was improved, but web reinforcement as used in these tests was not sufficient to prevent the occurrence of shear failures.

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Title No. 56-42

Artificial Carbonation of Concrete Masonry Units

By HENRY T. TOENNIES

Research to develop a process of artificial carbonation of concrete masonry units is described. A process of hastening carbonation (preshrinking) of masonry units would eliminate subsequent atmospheric carbonation and attendant shrinkage after the units were in a wall. Research was prompted by the feasibility of artificially carbonating concrete masonry units, using as a source of carbon dioxide the flue gases from combustion in steam boilers adjunct to steam curing.

Flue gas carbonation effected reductions in early-age drying shrinkage of masonry units. Reductions varied with carbonation time, temperature, and precarbonation moisture condition of units. Greatest reductions accompanied treatment at elevated temperatures immediately after completion of steam curing. Early-age shrinkage benefits due to carbonation were reflected in total shrinkage with time during 10 months of atmospheric exposure.

The effect of carbonation on other measured physical properties of concrete masonry appeared minor.

CARBONATION IS THE CHEMICAL COMBINING of carbon dioxide and the hydration products of portland cement. It has been of interest to manufacturers of concrete masonry since 1955 when Shideler[1] demonstrated that carbonation of masonry units is accompanied by their linear shrinkage. Concrete masonry units, predried to a relatively low moisture content, continued to shrink with prolonged exposure to the atmosphere due to carbonation by the small volume (0.02 to 0.03 percent) of carbon dioxide in the air. This slow carbonation by the carbon dioxide in the air and its attendant shrinkage may be a contributing factor to masonry wall cracking. Prior to 1955, expansion and contraction of masonry were attributed to temperature changes and to changes in unit moisture content. Shideler's findings added a new concept to the causes and possible correction of wall cracking. A process of hastening carbonation of masonry units might be developed which would "preshrink" them, thus eliminating atmospheric carbonation and attendant shrinkage after the units were in a wall (Fig. 1). This paper describes a project to develop such a process of artificial carbonation.

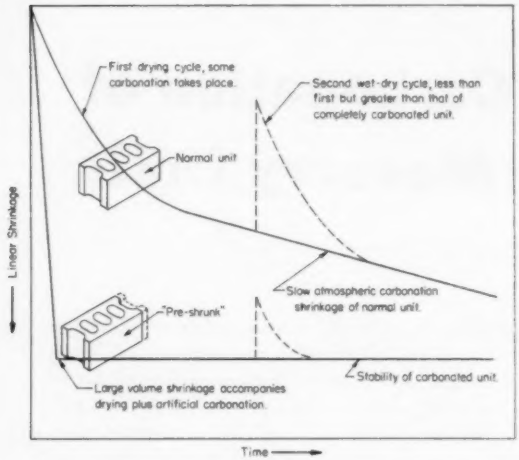
First drying cycle, some carbonation takes place.
Second wet-dry cycle, less than first but greater than that of completely carbonated unit.
Normal unit
Linear Shrinkage
Slow atmospheric carbonation shrinkage of normal unit.
"Pre-shrunk"
Stability of carbonated unit.
Large volume shrinkage accompanies drying plus artificial carbonation.
Time

Fig. 1—Preshrinking masonry with artificial carbonation

Artificial carbonation requires exposure of fully cured concrete masonry units to a greater concentration of CO_2 than that found in normal atmosphere. Early investigators of the subject have reported that carbonation cannot be induced by the addition of CO_2-generating materials to the concrete mix,[2] and that carbonation should be preceded by curing of the concrete.[3] Carbonation of unhydrated cement paste resulted in concrete chalking and crumbling.

A second requirement is that the concrete be receptive to artificial carbonation. Shideler found the moisture condition of the masonry units had an influence on their receptiveness. Units predried to a moisture condition in equilibrium with air at 50 percent relative humidity could be artificially carbonated, while those predried at 11 percent could not. This has been verified by the more recent work of Verbeck[4] who found that mortar specimens would carbonate less at 25 percent relative humidity than at 50 or 75 percent. Verbeck also reported little carbonation with specimens exposed to a relative humidity of 100 percent. This confirmed the work of Leber and Blakey[5] who found that artificial carbonation was retarded when the specimens were exposed to the carbon dioxide while in a wet condition. Thus it would appear that the optimum moisture condition for artificial carbonation is somewhere between the wet and the dry state. Receptiveness is further complicated by water which is produced during carbonation. A unit at optimum moisture condition at the beginning of carbonation will undergo an increase in moisture content as the reaction proceeds, and will reach a moisture condition which will inhibit further carbonation unless some means are provided for removal of this newly formed water. Other factors such as cement type and method of curing have been suggested as influencing

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receptiveness but as yet have not been investigated fully. In the program described here an attempt was made to minimize the effect of influencing factors by using materials believed to be receptive to carbonation.

The most obvious method of carbonating artificially would be to place masonry units in an air-tight room or container, evacuate the air from the container and replace it with pure or nearly pure carbon dioxide available from a commercial source. If the units were receptive, carbonation should proceed at an accelerated rate. The time required for complete carbonation would depend on receptiveness of the masonry units at the start of and during the exposure period. This method of treatment has received consideration and eventually could become a practical way to carbonate. However, it is fairly expensive. It requires approximately 1 lb of CO_2 to completely carbonate an 8-in. hollow concrete masonry unit. The price of commercially available CO_2 ranges from $0.035 to $0.07 per lb. It is doubtful whether the benefits derived from artificial carbonation would warrant so great an expenditure per unit.

A possible source of pure or nearly pure carbon dioxide is located at the concrete products plant. Most plants have steam boilers which produce steam for curing. The waste flue gases from these boilers normally contain from 10 to 18 percent carbon dioxide. One pound of CO_2 is produced during the steam curing of 6 to 8 concrete masonry units. Supplementary equipment might be installed to recover and utilize this waste CO_2 for carbonation. Although the total amount recovered would not be sufficient to completely carbonate all of production, it could be used to completely carbonate a portion of total production or to partially carbonate all of production. One concrete products plant is known to have installed this type of equipment.

Another possibility is to expose cured masonry units to the flue gases themselves, without prior recovery of the carbon dioxide. This method would require a minimum of equipment. All that would be needed are fans, ducts to carry the flue gases, and auxiliary cooling equipment. This third possibility prompted the investigation described. The purpose of the investigation was to determine the feasibility of carbonating masonry units with the carbon dioxide gas contained in flue gases from the combustion of carbonaceous fuels. The investigation required construction of a pilot artificial carbonation chamber which produced flue gases con-

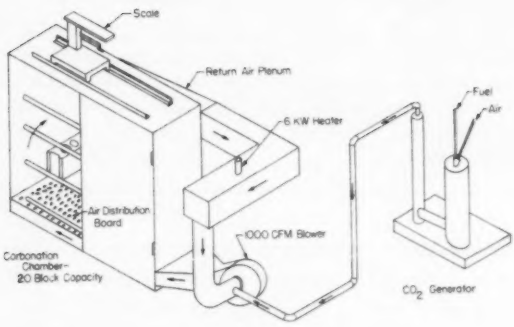


Fig. 2—Pilot artificial carbonation chamber

taining a normal concentration of carbon dioxide, the exposure of concrete masonry to these flue gases and to pure CO_2, and the measurement of carbonation extent and effect on various types of masonry units.

Apparatus

Artificial carbonation with flue gas was accomplished in the specially constructed apparatus shown in Fig. 2. It consists of a reasonably air-tight chamber large enough to hold 20 regular 8 x 8 x 16-in. concrete masonry units. Flue gases are produced by a gas combustion burner shown at the far right of the chamber. This burner is equipped with a water spray which cools the flue gases to about 100 F. The cooled gases are then piped into the system, entering at a point in the circulating air duct. An electrical heating element inside the duct permits reheating of the gases to any desired temperature. The gases then pass through a large blower and enter the carbonation chamber at the bottom right corner. Inside the chamber, the gases are distributed through a false bottom containing numerous drilled holes. They then pass upward through the chamber, around and over the masonry test specimens which are supported on open racks with their cores vertical. Finally, the gases are returned through an air plenum leading back to the circulating air duct. The top row of specimens were equipped with metal hangers so they could be weighed without opening the chamber door. Specimen temperature, carbonation temperature, and the air temperature inside the return air duct were measured by thermocouples connected to a remote reading instrument. Other equipment included a pressure manometer mounted on the cabinet exterior, a dew point measuring apparatus, and an apparatus for measuring the CO_2 concentration in the chamber. Temperatures, weights, cabinet pressure, relative humidity, and CO_2 concentration were logged hourly during each carbonation trial run.

Carbonation with pure CO_2 was accomplished with the apparatus shown in Fig. 3. It consists of a steel autoclave equipped with a fan for maintaining air circulation and a desiccant for maintaining relative humidity. Carbon dioxide was piped into the autoclave from commercially available 50-lb cylinders at a rate of 5 to 10 lb per day.

Test program

The investigation was divided into three series of tests. Table 2 gives descriptive information on the three series, while Table 1 lists mix composition and curing information of the hollow-core 8 x 8 x 16-in. units which were purchased locally

for use as test specimens. Series I carbonation was with flue gas, Series II with pure carbon dioxide, and Series III was a combination of the two previous series, i.e., specimens were carbonated first with flue gas, tested, then further carbonated with pure carbon dioxide.

Series I, an exploratory series, included seven flue gas carbonation runs on cinder aggregate specimens previously cured with steam at atmospheric pressure. Variables investigated were: (1) effect of sample age and moisture condition on extent of artificial carbonation, (2) effect of carbonation temperature, and (3) effect of carbonation time. Each run was for 9 hr, but representative samples were taken from the carbonation chamber after 3, 6, and 9 hr of exposure. Carbonation Runs A, B, and C were made on freshly cured specimens having an initial moisture content of approximately 77 percent total H_2O absorption. Temperatures for the three runs were respectively 100, 150, and 200 F. Runs E and F were on specimens previously dried to a moisture condition in equilibrium with air at 75 percent relative humidity and 73 F. Initial moisture content of these specimens was approximately 8 percent of total H_2O absorption. Temperatures for Runs E and F were 100 and 150 F. Specimens for Runs G and H were predried also, but to a lower moisture condition in equilibrium with air at 50 percent relative humidity. Their initial moisture content was approximately 6 percent. Carbonation temperatures for Runs G and H were 100 and 200 F.

Following carbonation, Series I specimens were conditioned 10 days in air at 50 percent relative humidity then tested for compressive strength, unit weight, and linear shrinkage due to drying from saturation to an equilibrium condition in air at 50 percent relative humidity and 73 F. A chemical test was made on some of the specimens to determine their carbon dioxide content as shown in Table 3. Since the cinder aggregate contained a negligible amount of calcareous material, it was believed measured increase in CO_2 content would indicate extent of carbonation. All tests except drying shrinkage and CO_2 content were made in accordance with applicable ASTM standards using three specimen samples rather than five. The effects of carbonation were observed by comparing test results on carbonated specimens with those of noncarbonated control specimens.

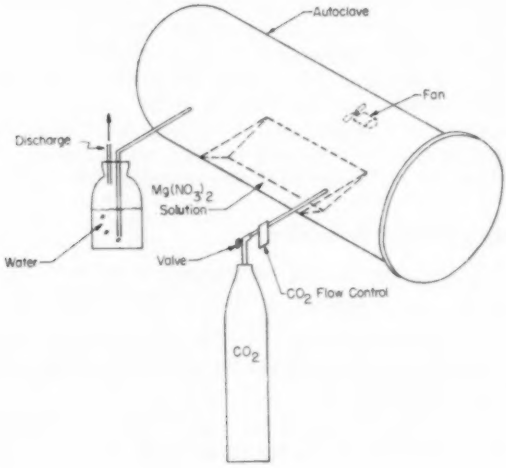


Fig. 3—Pure CO_2 carbonation apparatus

TABLE 1—MIX COMPOSITION AND CURING INFORMATION OF 8 x 8 x 16-IN. HOLLOW MASONRY UNITS

Series	I and II	III	
Principal aggregate*	Steam boiler cinders	Expanded shale	Expanded slag
Mix: portland cement (Type I), lb	564	500	500
Aggregate, lb	3000	4000	3100
Proportions by weight	1:5.3	1:8.0	1:6.2
Steam curing: maximum temperature, F	165	170	160
Time, hr.	16	16	16
Yield of block per bag of cement	22	34	28

*Aggregate consisted entirely of those shown except for expanded shale batch which contained 400 lb of limestone sand and 3600 lb of shale.

Series II consisted of three samples of cinder units carbonated with pure dioxide instead of flue gases. The series provided information on complete carbonation for comparison with the partial carbonation effected in Series I. The three trial Runs J, K, and M differed only in sample initial moisture condition. Run J specimens had a high moisture content and were carbonated and dried simultaneously. Run K specimens were predried in air at 75 percent relative humidity, and Run M specimens were predried at 50 percent. During carbonation with pure CO_2 the atmosphere was maintained at a relative humidity of 53 percent with a magnesium nitrate solution.

Series III was an extension of exploratory Series I. Samples of two different aggregate types (expanded shale and expanded slag) were carbonated with flue gas. Optimum conditions for artificial carbonation were selected based on Series I tests except that exposure periods were arbitrarily increased to 9 and 18 hr. Run N specimens were hollow-core 8 x 8 x 16-in. expanded shale units previously cured with steam at atmospheric pressure. Specimens were carbonated at 150 F immediately after steam curing. Run P specimens were of similar size and cure, but of expanded slag aggregate. They too were carbonated immediately after steam curing, but instead of maintaining a constant carbonation temperature the relative humidity inside the chamber was held between 35 and 55 percent. This required hourly changes in the carbonation temperature which varied from 160 to 100 F during the 18-hr run. After physical and chemical tests, Series III specimens were exposed to pure carbon dioxide. Thus the procedure for Series III was a combination of that employed in Series I and Series II.

Results and discussion

The scope of this investigation permits evaluation of artificial carbonation in a number of ways. These include:

1. Comparison of drying shrinkage of specimens (a) carbonated with flue gases, (b) carbonated with pure carbon dioxide, and (c) noncarbonated control specimens.

2. Comparison of the long-time dimensional stability of carbonated and noncarbonated control specimens.

3. Comparison of other physical properties, such as, compressive strength, absorption, and unit weight of carbonated and control specimens.

4. Observation of changes in specimen weight, length, and CO_2 content during carbonation with either pure carbon dioxide or flue gases.

Comparing physical properties of carbonated and noncarbonated control specimens bears directly on the program objective which was to learn if concrete masonry units can be benefited by carbonation with flue gases. Observing specimen physical and chemical changes during carbonation is of interest because of the present limited published information on the subject. Some changes observed during carbonation

TABLE 2—SAMPLE IDENTIFICATION AND TRIAL RUN SCHEDULE

Run	Aggregate	Precarbonation drying treatment	Sample	Carbonation conditions			
				Time	Temperature, F	Relative humidity, percent*	CO_2 percent
Series I—Flue gas							
A	Cinder	None	A3-100 A6-100 A9-100	3 hr 6 hr 9 hr	100	70	5.6
E	Cinder	73 F - 75 percent relative humidity	E3-100 E6-100 E9-100	3 hr 6 hr 9 hr	100	35	7.3
F	Cinder	73 F - 75 percent relative humidity	F3-150 F6-150 F9-150	3 hr 6 hr 9 hr	150	14	7.3
G	Cinder	73 F - 50 percent relative humidity	G3-100 G6-100 G9-100	3 hr 6 hr 9 hr	100	30	6.6
H	Cinder	73 F - 50 percent relative humidity	H3-200 H6-200 H9-200	3 hr 6 hr 9 hr	200	8	8.0
B	Cinder	None	B3-150 B6-150 B9-150	3 hr 6 hr 9 hr	150	30	6.8
C	Cinder	None	C3-200 C6-200 C9-200	3 hr 6 hr 9 hr	200	10	7.0
Series II—Pure CO_2							
J	Cinder	None	J-CO	125 days	75	Not measured	
K	Cinder	73 F - 75 percent relative humidity	K-CO	30 days	75		
M	Cinder	73 F - 50 percent relative humidity	M-CO	46 days	75		
Series III—Flue gas plus pure CO_2							
N	Shale	None	N9-150 N18-150	9 hr 18 hr	150	25	9.3
P	Slag	None	P9 P18	9 hr 18 hr	†	40	7.9

*Average of hourly measurements.
†Temperature varied from 100 to 160 F while relative humidity was maintained between 35 and 55 percent.

have an influence on conclusions relating to the program objective and will be considered first in the discussion which follows.

Table 3 lists changes caused by flue gas carbonation at various temperatures, relative humidities, and exposure periods on samples having different initial moisture conditions. Most samples lost weight and shrank during carbonation, due to their high initial moisture content and/or the high carbonation temperatures. The foremost indication of

TABLE 3—SAMPLE CHANGES DURING CARBONATION WITH FLUE GAS

Run	Sample	Observed changes in sample		
		Weight, percent	Length, percent	CO_2 content, percent
A	A3-100	—0.37	0.000	Not measured
	A6-100	—0.93	0.000	
	A9-100	—1.38	—0.017	
B	B3-150	—2.56	+0.002	Not measured
	B6-150	—4.66	—0.007	
	B9-150	—6.46	—0.016	
C	C3-200	—4.94	—0.007	Not measured
	C6-200	—8.03	—0.031	
	C9-200	—8.75	—0.042	
E	E3-100	+0.69	+0.002	—
	E6-100	+1.13	—0.003	—
	E9-100	+1.34	—0.016	+1.00
F	F3-150	—0.16	+0.002	—
	F6-150	—0.69	—0.023	—
	F9-150	—0.86	—0.029	+0.62
G	G3-100	+0.61	+0.002	—
	G6-100	+1.17	—0.008	—
	G9-100	+1.21	—0.008	—0.07
H	H3-200	—0.83	—0.030	—
	H6-200	—1.65	—0.024	—
	H9-200	—1.80	—0.030	—0.70
N	N9-150	—4.16	+0.004	+0.91
	N18-150	—5.92	—0.012	+1.05
P	P9	—3.84	—0.003	+1.54
	P18	—2.78	—0.007	+3.43

TABLE 4—SAMPLE CHANGES DURING AIR DRYING AND CARBONATION WITH PURE CARBON DIOXIDE

Run	Sample	Observed changes in sample				
		During air drying		During carbonation with pure CO_2		
		Weight, percent	Length, percent	Weight, percent	Length, percent	CO_2 content, percent
J*	J-CO			—1.43	—0.038	+6.14
K	K-CO	—6.90	—0.047	+2.70	—0.008	+3.06
M	M-CO	—6.80	—0.052	+3.64	—0.029	+4.98
N†	N9-CO	—9.50	—0.040	+1.73	—0.032	+0.36
	N18-CO	—10.70	—0.034	+1.42	—0.027	+2.31
P†	P9-CO	—8.70	—0.043	+2.55	—0.029	+1.46
	P18-CO	—8.82	—0.037	+1.80	—0.024	+1.04

*Carbonated and dried simultaneously.
†Previously carbonated with flue gas.

carbonation, i.e., shrinkage accompanied by weight gain, was noted in but two runs (E and G). It is reasonable to assume that some carbonation took place during these two runs. That all other runs resulted in weight loss does not mean necessarily that these runs resulted in no carbonation. Weight gain due to carbonation may have been obscured by moisture loss at the high carbonation temperatures. The gain in CO_2 content during Runs F, N, and P is evidence of carbonation even though each of these runs resulted in a loss of weight. It will be shown in the following paragraphs that an increase in CO_2 content is a better indication of carbonation than is shrinkage.

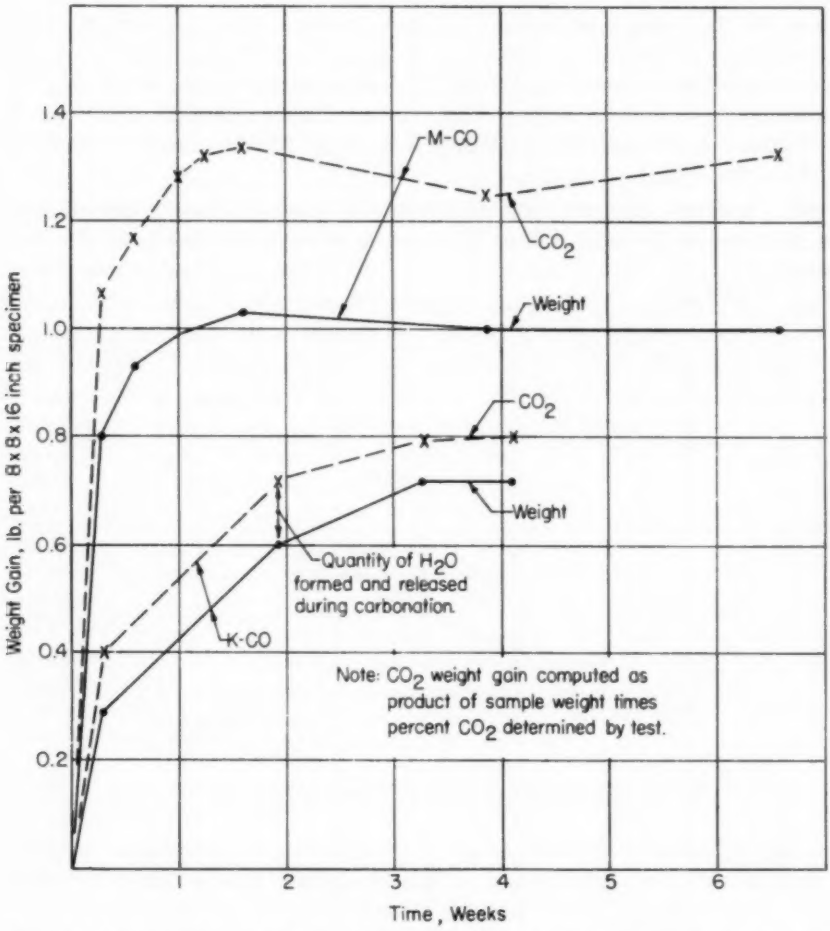


Fig. 4—Increases in weight and CO_2 content during carbonation with pure carbon dioxide

Table 4 lists changes observed on six samples air dried and subsequently carbonated in pure carbon dioxide, and on one sample simultaneously dried and carbonated (Series II). Runs J, K, and M were made with cinder aggregate specimens similar to those used in the Series I tests. Runs N and P were made with shale and slag aggregate units previously carbonated with flue gas. The changes during drying reported for Runs N and P are from the early-age shrinkage tests made after carbonation with flue gas.

All of the predried samples shrank and gained weight during carbonation with pure carbon dioxide (Table 4). Shrinkage was greater for samples predried at 50 percent relative humidity (Runs M, N, and P) than for the sample predried at 75 percent (Run K). All samples were carbonated at a relative humidity of 53 percent. The sample dried and carbonated simultaneously (Run J) had a weight loss, indicating its moisture loss due to drying was greater than its weight gain due to carbonation. All samples carbonated in pure carbon dioxide increased in CO_2 content.

Fig. 4 shows increase in weight and CO_2 content during carbonation of samples M-CO and K-CO. There is a relationship between the two measurements; either can be used to indicate extent of carbonation. This relationship between gain in carbon dioxide content and weight

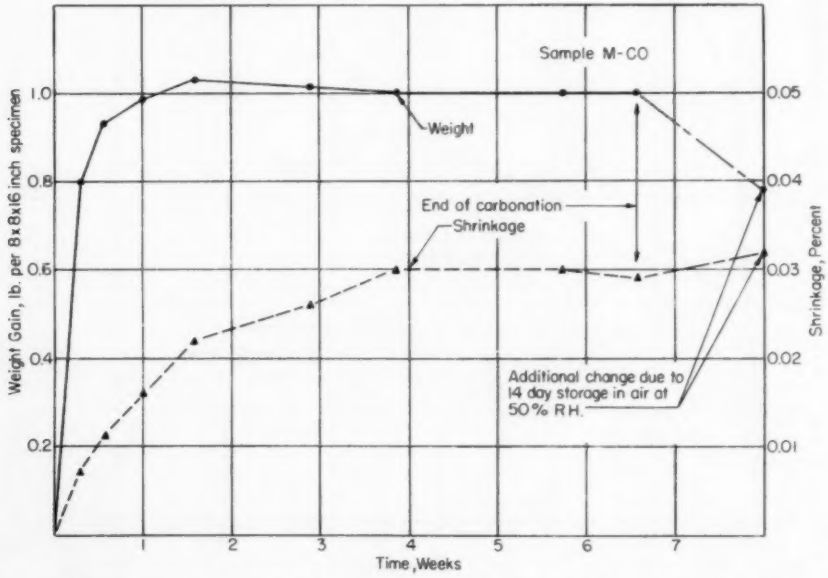


Fig. 5—Weight gain and shrinkage of Sample M-CO during carbonation with pure carbon dioxide

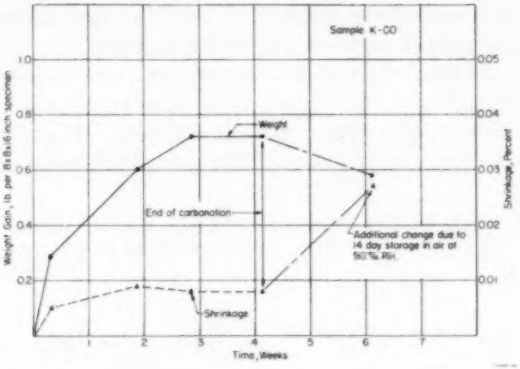


Fig. 6 — Weight gain and shrinkage of Sample K-CO during carbonation with pure carbon dioxide

gain has been pointed out previously by Verbeck[4] for carbonated cement pastes containing different amounts of fly ash. A similar relationship cannot be shown for Sample J-CO because of its high initial moisture content and subsequent weight loss during simultaneous carbonation and drying. The difference between CO_2 content increase and weight increase shown in Fig. 4 is due presumably to the release of newly formed water during carbonation.

Shrinkage during carbonation appears to have less relationship to extent of carbonation than does weight and CO_2 gain. This is illustrated in Fig. 5 and 6 where shrinkage and weight gain are plotted against carbonation time. Note in Fig. 5 that Sample M-CO attained maximum weight gain (extent of carbonation) after 11 days of exposure, but continued to shrink for a total of 4 weeks. Sample K-CO, Fig. 6, behaved in a different fashion, reaching maximum shrinkage in 13 days but requiring 20 days before reaching maximum weight gain. Both samples lost weight and shrank during 14 days of subsequent air storage at 50 percent relative humidity. Based on these limited tests, the amount of shrinkage a concrete masonry unit undergoes during carbonation with pure CO_2 appears unrelated to the extent of carbonation except indirectly as carbonation affects the ability of the masonry unit to retain moisture. The relationship between masonry shrinkage and affinity for H_2O will be discussed later.

EFFECT OF CARBONATION ON PHYSICAL PROPERTIES

Tests were conducted to determine the effects of carbonation on the concrete masonry specimens. Results of these tests are given in Table 5 along with similar data for control (noncarbonated) specimens. Tests included compressive strength, absorption, unit weight, and drying shrinkage on representative samples of three specimens each. Generally

the tests were conducted on both carbonated and control specimens 10 days after carbonation of the former.

Carbonation, with flue gas or pure carbon dioxide, had relatively insignificant effects on specimen compressive strength, absorption, and unit weight. Flue gas carbonation (Fig. 7) increased the compressive strength of the Series III, expanded shale and slag specimens, but had

TABLE 5—PHYSICAL PROPERTIES OF MASONRY UNITS AFTER EXPOSURE TO FLUE GAS AND/OR PURE CARBON DIOXIDE

Run	Sample	Compressive strength, gross area, psi	Absorption,* lb per cu ft	Unit weight,* lb per cu ft	Shrinkage from drying	
					Percent	Carbonated / Control
A	A3-100	720	14.1	81.5	0.047	1.07
	A6-100	720	14.0	82.1	0.047	1.07
	A9-100	775	14.7	81.6	0.048	1.09
B	B3-150	640	13.5	81.6	0.037	0.79
	B6-150	585	13.2	81.7	0.028	0.60
	B9-150	640	12.9	82.8	0.022	0.47
C	C3-200	645	13.2	81.3	0.033	0.70
	C6-200	595	13.4	81.4	0.026	0.55
	C9-200	565	13.3	81.2	0.026	0.55
E	E3-100	810	13.6	81.3	0.044	1.00
	E6-100	850	13.1	82.8	0.038	0.86
	E9-100	860	12.8	82.7	0.037	0.84
F	F3-150	705	13.3	81.5	0.038	0.86
	F6-150	745	13.3	82.1	0.034	0.77
	F9-150	815	13.5	82.2	0.035	0.80
G	G3-100	845	15.2	78.7	0.043	0.98
	G6-100	880	15.0	80.0	0.034	0.77
	G9-100	850	14.7	80.6	0.032	0.73
H	H3-200	900	15.1	78.1	0.039	0.89
	H6-200	875	15.1	78.4	0.038	0.86
	H9-200	855	16.2	77.8	0.042	0.95
Control 1†		755	13.8	80.8	0.044	1.00
Control 2†		610	14.2	80.0	0.047	1.00
N	N9-150	2150	—	—	0.040	0.70
	N18-150	2100	—	—	0.034	0.60
P	P9	1090	—	—	0.043	0.91
	P18	1180	—	—	0.037	0.79
Control 4†		1705	—	—	0.057	1.00
Control 5†		990	—	—	0.047	1.00
J	J-CO	1080	11.9	88.4	0.020	0.54§
K	K-CO	905	12.3	87.8	0.022	0.60§
M	M-CO	970	12.1	88.2	0.020	0.54§
	Control	585 (1d) 845 (60d) 850 (125d) 880 (150d)			0.057‡ 0.037	1.00
P-CO	P9-CO	1150	14.2	98.8	0.019	0.51
	P18-CO	1255	12.7	101.0	0.019	0.51
Control 5†		980	13.9	95.1	0.037	1.00
N-CO	N9-CO	1860	11.5	95.5	0.026	0.72
	N18-CO	1590	11.6	95.0	0.021	0.58
Control 4†		1845	12.3	92.0	0.036	1.00

*Absorption and unit weight based on saturated and inundated weights at start of shrinkage test and oven dry weight after test.
†Control specimens were not subjected to artificial carbonation.
‡Shrinkage during storage (60 days) prior to shrinkage test.
§Based on shrinkage of 0.037 percent for control specimens.

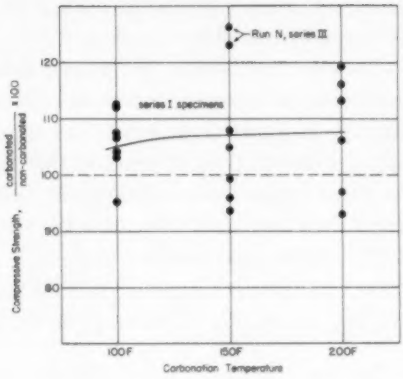


Fig. 7—Effect of flue gas carbonation on compressive strength

minor effect on the Series I, cinder specimens. Greatest strength increase occurred with the expanded shale specimens (Run N) and amounted to approximately 400 psi. Strength increases accompanied carbonation with pure CO_2 in most cases (Table 5) but the compressive strength of the expanded shale specimens (Run N-CO) was less after exposure to pure CO_2 than it had been after flue gas carbonation. Based on these data it is questionable whether carbonation has a significant beneficial effect on the compressive strength of concrete masonry.

Drying shrinkage was altered by both flue gas carbonation and carbonation with pure carbon dioxide. Carbonation resulted in an average reduction of 43 percent in the drying shrinkage of eight samples exposed to pure CO_2. Reduction is based on a comparison with the measured shrinkage of noncarbonated control specimens during their second drying cycle and would be greater if based on shrinkage of control units during their first drying cycle.

Note in Table 5 the three cinder aggregate samples carbonated with pure CO_2 (Runs J, K, and M) each had approximately the same drying shrinkage. The same is true of the expanded slag units, (Run P-CO) and to a lesser degree of the expanded shale (Run N-CO). None of the changes observed during carbonation (Table 4) account for this sameness in the drying shrinkage of like-aggregate samples having different degrees of apparent carbonation. Although weight gain appeared related to CO_2 gain during carbonation of the two cinder aggregate samples K-CO and M-CO (Fig. 4) the indicated degree of carbonation of the two samples was not the same. It would seem that reductions in drying shrinkage are not proportional to weight gain, CO_2 gain, or shrinkage occurring during carbonation.

Linear drying shrinkages of samples carbonated with flue gas are shown in Table 5. The effect of carbonation is indicated by the ratio

of carbonation sample shrinkage to that of the control specimens. These ratios are plotted in Fig. 8.

Greatest reduction in drying shrinkage was caused by flue gas carbonation at temperatures of 150 and 200 F immediately after steam curing (Runs B, C, and N). The drying shrinkage of the cinder aggregate sample B9-150 was 47 percent that of a companion noncarbonated sample. By comparison, this is a greater reduction in drying shrinkage than that due to carbonation of cinder units with pure CO_2 (Runs J, K, and M).

The average effects of exposure temperature and time on the drying shrinkage of all samples carbonated with flue gas are shown in Fig. 9a and 9b, while Fig. 9c shows the average effect of precarbonation drying,

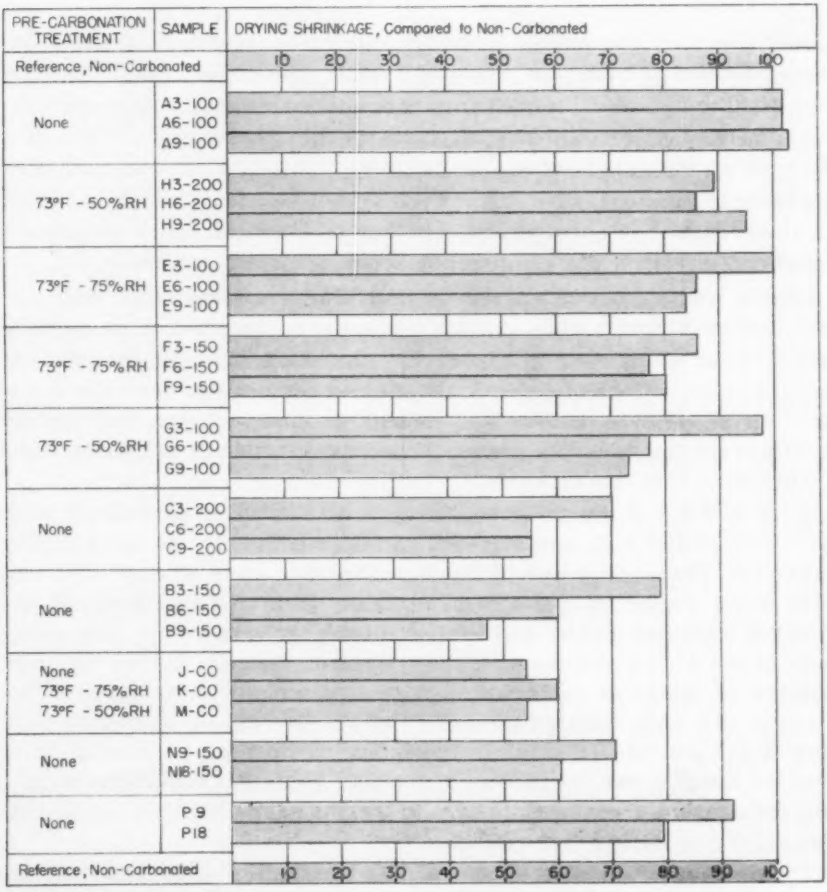
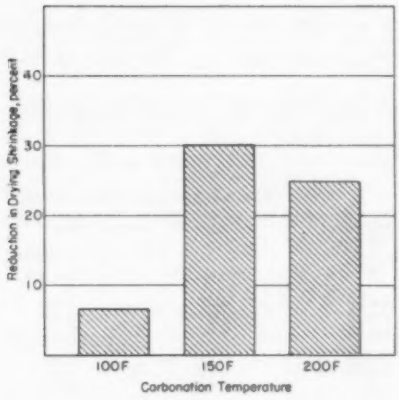
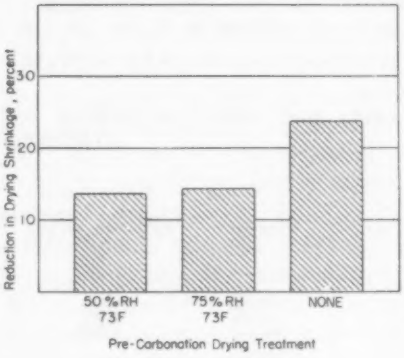


Fig. 8—Effect of flue gas carbonation on drying shrinkage

or initial moisture condition of sample. Considering all samples, reductions in drying shrinkage were greatest for samples carbonated at 150 F (Fig. 9a) immediately after their steam curing (Fig. 9c) and were generally greater due to longer exposures to the flue gases (Fig. 9b).

After they had reached a stable condition of dryness, Series I shrinkage specimens were dried further at 220 F and subsequently stored outside of the laboratory for approximately 10 months. Prolonged outdoor exposure permitted further carbonation by the CO_2 in the atmosphere. Presumably masonry units which have been artificially carbonated, "preshrunk," should be dimensionally stable during prolonged exposure to the atmosphere (see Fig. 1). Following prolonged exposure, specimens were returned to the laboratory, resaturated, and again tested for drying shrinkage. Table 6 lists specimen weight and length changes noted between the time the samples were at a stable condition at completion of the first shrinkage test and when they were again at a stable condition at the end of the second test 10 months later. All samples gained weight and shrank during the 10-months. Complete linear stability was





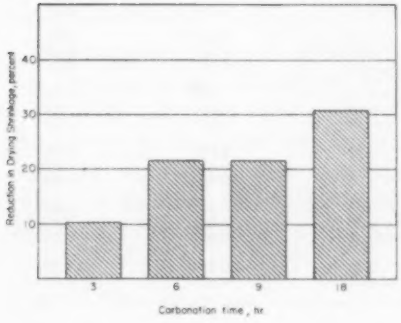


Fig. 9a (upper left)—Effect of carbonation temperature on drying shrinkage. (b) (left) — Effect of carbonation time on drying shrinkage. (c) (upper right)—Effect of precarbonation drying on drying shrinkage of carbonated units

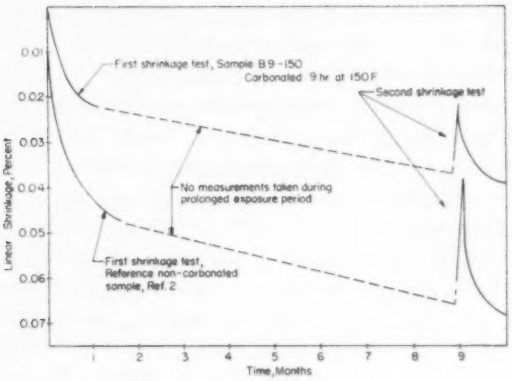

First shrinkage test, Sample B9-150
Carbonated 9 hr at 150 F
Second shrinkage test
No measurements taken during prolonged exposure period
First shrinkage test, Reference non-carbonated sample, Ref. 2
Linear Shrinkage, Percent
Time, Months

Fig. 10—Effect of flue gas carbonation on long-time volume — stability of masonry units

not obtained by flue gas carbonation. In most cases the amount of additional shrinkage was less for the carbonated than for the non-carbonated control specimens.

Total shrinkage due to two cycles of wetting and drying and 10 months of atmospheric exposure are shown also in Table 6. The effect of flue gas carbonation is shown by the ratio of carbonated to noncarbonated specimen total shrinkage from saturation at the start of the first shrink-

TABLE 6—CHANGE IN SAMPLE WEIGHT AND LENGTH WITH PROLONGED ATMOSPHERIC EXPOSURE

Run	Sample	Change observed between end of first and second shrinkage tests		Total shrinkage from initial saturation (first test) to dry condition (second test)	
		Length, percent	Weight, lb	Percent	Carbonated / Control
A	A3-100	—0.026	+0.38	0.073	1.14
	A6-100	—0.024	+0.35	0.071	1.11
	A9-100	—0.022	+0.38	0.070	1.10
B	B3-150	—0.026	+0.39	0.063	0.89
	B6-150	—0.022	+0.36	0.050	0.71
	B9-150	—0.017	+0.29	0.039	0.55
C	C3-200	—0.027	+0.51	0.060	0.85
	C6-200	—0.017	+0.50	0.043	0.61
	C9-200	—0.016	+0.49	0.042	0.59
E	E3-100	—0.021	+0.35	0.063	0.99
	E6-100	—0.013	+0.26	0.051	0.80
	E9-100	—0.009	+0.20	0.046	0.72
F	F3-150	—0.018	+0.33	0.056	0.88
	F6-150	—0.013	+0.25	0.047	0.74
	F9-150	—0.013	+0.19	0.048	0.75
G	G3-100	—0.013	+0.39	0.058	0.91
	G6-100	—0.009	+0.32	0.043	0.67
	G9-100	—0.012	+0.28	0.044	0.69
H	H3-200	—0.018	+0.41	0.057	0.89
	H6-200	—0.013	+0.38	0.051	0.80
	H9-200	—0.012	+0.37	0.054	0.85
	Control 1	—0.020	+0.49	0.064	1.00
	Control 2	—0.024	+0.61	0.071	1.00

age test to equilibrium dryness (50 percent relative humidity, 73 F) at the end of the second test, 10 months later. Numerically, differences in total shrinkage of carbonated and noncarbonated specimens (Table 6) are about the same as shown by early age drying shrinkage tests (Table 5). However, the ratio of carbonated to noncarbonated total shrinkage increased due to an additional equal amount of shrinkage of both specimen types during the 10 months of atmospheric exposure. Fig. 10 shows the time-shrinkage curves for Sample B9-150 and the noncarbonated control sample. It appears the improvement in drying shrinkage resulting from flue gas carbonation is permanent even though specimens undergo a normal amount of atmospheric carbonation shrinkage during prolonged exposure.

RELATIONSHIP OF DRYING SHRINKAGE TO RESIDUAL MOISTURE

After they had reached a stable condition of dryness at 50 percent relative humidity and 73 F, Series I shrinkage specimens were dried at 220 F to determine their residual shrinkage and moisture content. Earlier tests* had indicated a relationship between shrinkage during drying and the quantity of moisture retained in the unit on completion of drying (residual moisture). This relationship has more recently been shown by Verbeck[4] who compared length change during drying with the amount of water vapor adsorbed by cement pastes at 36 percent relative humidity. The amount of vapor adsorbed is reported[6] to be a measure of the amount and physical character of the products of cement hydration. Determining the residual moisture retained at 50 percent relative humidity by oven drying at 220 F is a less refined procedure than that employed by Verbeck and others with cement pastes, but results appear to agree with good uniformity. Fig. 11 shows the relationship between measured shrinkage and residual moisture content as determined by oven drying 23 samples from Series I after two different shrinkage tests, and three Series II samples after one shrinkage test. The relationship is fairly consistent; unit shrinkage during drying is directly proportional to residual water content at equilibrium.

The determination of residual moisture content is a simple method of measuring the shrinkage characteristics of concrete masonry. Undoubtedly, such factors as type and amount of aggregate also influence drying shrinkage, as does the temperature and relative humidity of the drying atmosphere. Higher drying temperature and lower relative humidities result in reduced residual moisture and greater drying shrinkage. It is believed, however, that with further study, and research, residual mois-

*Unpublished test data of the National Concrete Masonry Association.

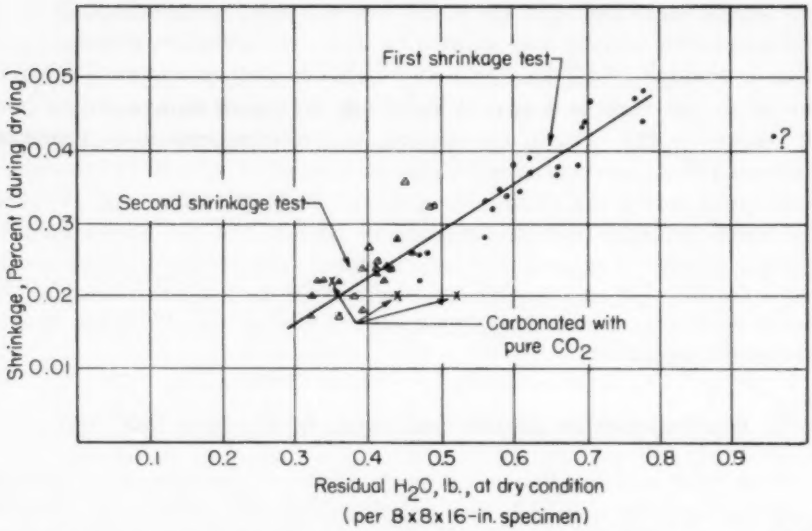


Fig. 11—Relationship between drying shrinkage and residual moisture content after drying of masonry units

ture content (or water sorption) may form a rational basis for predicting the drying shrinkage of concrete masonry.

SUMMARY

1. Early-age drying shrinkage of concrete masonry units was altered by both carbonation with flue gas and carbonation with pure CO_2. The drying shrinkage of units carbonated with pure CO_2 was reduced an average of 43 percent. The drying shrinkage of units carbonated with flue gas was reduced, reductions varying with carbonation time, temperature, and precarbonation moisture condition of units.

The effect of flue gas carbonation on early-age drying shrinkage was greatest (reduction in drying shrinkage of up to 53 percent) when concrete masonry units were treated at temperatures of 150 and 200 F immediately after steam curing. Flue gas carbonation was least effective in reducing early-age drying shrinkage when units were treated at 100 F immediately after steam curing.

Considering all samples, reductions in early-age drying shrinkage due to flue gas carbonation were greatest when units were treated at 150 F, (Fig. 9a); were greatest when units were treated immediately after steam curing, (Fig. 9c); and were generally greater due to longer exposure to the flue gases (Fig. 9b).

2. The effect of flue gas carbonation on early-age drying shrinkage was reflected in the total shrinkage of samples after 10 months exposure

to the atmosphere and two drying shrinkage tests. Numerically, differences in total shrinkage of carbonated and noncarbonated specimens were about the same at the end of a shrinkage test after 10 months atmospheric exposure as they had been at the end of a shrinkage test at the beginning of the period.

3. The effect of carbonation, with pure CO_2 or flue gas, on concrete masonry compressive strength, absorption, and unit weight was minor.

4. Both weight gain and gain in CO_2 content could be used as measures of carbonation extent of predried masonry units carbonated with pure CO_2 (Fig. 4). Shrinkage during carbonation was unrelated to extent of carbonation with pure CO_2 (Fig. 5 and 6).

5. Residual moisture content after drying at 50 percent relative humidity and 73 F was related to the shrinkage characteristics of concrete masonry. Shrinkage of atmospheric-pressure steam cured cinder aggregate units was directly proportional to their residue moisture content after drying (Fig. 11).

ACKNOWLEDGMENTS

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Title No. 56-43

Distribution of Torsion and Bending Moments in Connected Beams and Slabs

By M. A. GOUDA

Presents a method for determining torsion and bending moments in concrete beams and slabs connected monolithically. Solution of the problem is based on the elastic torsion theory where the effect of the torsional rigidity of beams and slabs is taken into account. Beams are assumed to be homogeneous with their ends rigidly fixed.

Different cases investigated include: outer beam subjected to a cantilever moment, marginal beam, effect of beam end rotation, and effect of beam deflection.

Formulas and practical curves are given for a direct determination of the torsion in the beams and the moments in the slabs.

■ IN BEAMS SUPPORTING A cantilever slab on one side and an inner slab on the other, the effect of the torsional resistance of the beam is generally neglected when calculating the actual moment to which the inner slab is subjected. The presence of this beam restrains, by an unknown amount, the bending of the cantilever slab. However, the inner slab is generally designed to resist all the external cantilever moment by considering the supporting beam as a knife edge support.

Also in beams supporting a slab on one side only, such as marginal or spandrel beams, the fact is overlooked that the presence of the floor slab prevents free rotation of the spandrel beam. This means that the connected slab will create an unknown restraint against the twisting of the beam.

Moreover, in office practice beams supporting a cantilever slab and an inner slab are designed for bending moment only, disregarding any twisting moment. The effect of the twisting moment is supposed to be covered by the factor of safety but this is not always a wise or correct procedure.

M. A. Gouda is assistant professor of structural engineering at Alexandria University, Egypt. Dr. Gouda received his MS in civil engineering from Columbia University, in 1947 and in 1951 he obtained his PhD at London University.

The object of this work is to present a method for the analyzing and determining the actual stresses in beams and slabs monolithically connected taking into account the effect of the torsional rigidity of each on the other.

To simplify the mathematical work of the problem the following assumptions are made:

(1) Beams and slabs are assumed homogeneous to allow the application of the torsion elastic theory.
(2) Ends of beams are assumed rigidly fixed.

In the investigation which follows, the theory is developed from a simple equation of equilibrium by considering a short length of beam subjected to an external moment m_o (if applied), a resisting torque T and the slab restraint moment m. Since the resisting moment m and torque T can be expressed in terms of the angle of deformation ϕ, a differential equation of the second degree may be derived in terms of the slab moment m as the variable. The solution of this equation will give the values of the unknown forces m and T.

CASE 1: BEAM CONNECTED TO AN INNER SLAB AND SUBJECTED TO AN EXTERNAL MOMENT m_o

Fig. 1 (a) and (b) show the plan and cross section of a beam of span l, breadth b, and over-all depth d connected to an inner slab of a uniform thickness t and span S. The beam is subjected to an external moment m_o from a cantilever slab as shown.

Assumptions

(a) Ends of beam are rigidly fixed.
(b) Beam is connected to a loaded cantilever on one side and to an unloaded inner slab ($l \times S$) on the other.
(c) Outer slab edge is simply supported.

General differential equation and its solution

Considering the equilibrium of an elementary strip of width dx cut by planes I, II, and III (Fig. 1) and subjected to forces as shown in Fig. 2 (a) and (b) we get:

$$m_o - m = \frac{dT}{dx} \qquad (1)$$

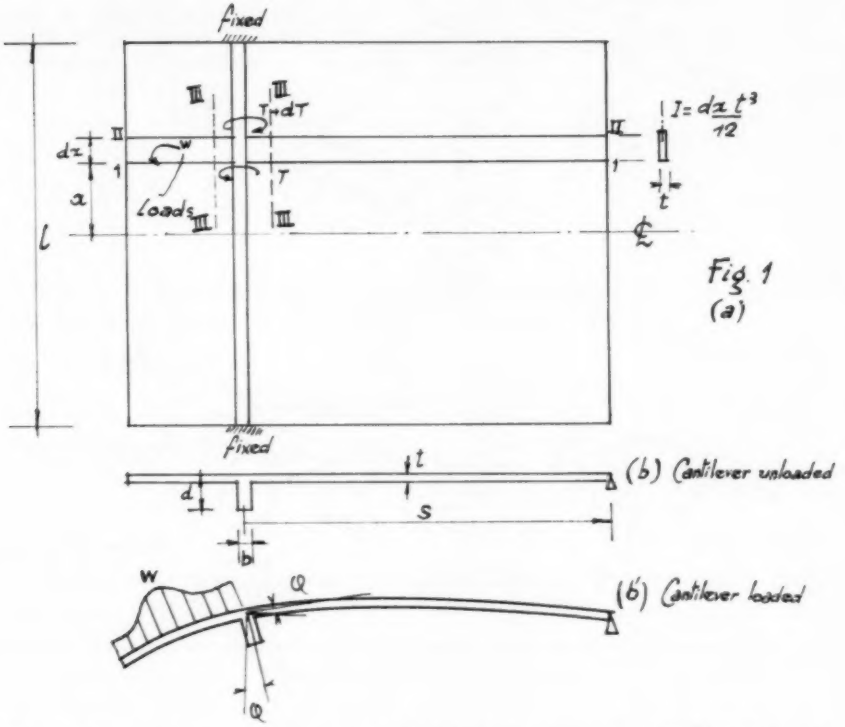

Fig. 1—Plan and cross section of a slab showing elementary strip with the cantilevered portion both loaded and unloaded

From the torsion theory: The angle of twist per unit length in the case of a rectangular cross section is given by:

$$\frac{d\phi}{dx} = \frac{T}{C} \qquad (2)$$

where C = torsional rigidity of beam $= \beta G b^3 d$ is a constant given in Table 1.

From Eq. (1) and (2),

$$C \frac{d^2\phi}{dx^2} = m_o - m \qquad (3)$$

TABLE 1

$\frac{d}{b}$	1	1.5	1.75	2	2.5	3	4	6	8	10
β	0.141	0.196	0.214	0.229	0.249	0.263	0.281	0.299	0.307	0.313

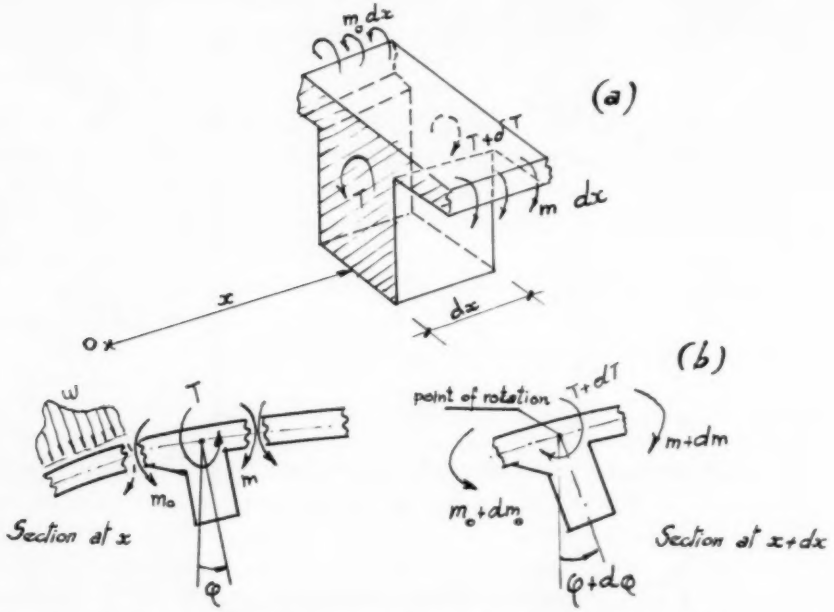

Fig. 2—Elementary strip of beam showing torsional effects on cantilever on one side and inner slab on the other

Referring to Fig. 3, angle of rotation

$$\phi = -\frac{(m\,dx\,S)}{3\,EI} = -\frac{4\,m\,S}{Et^3} \qquad (4)$$

Substituting in Eq. (3) from Eq. (4):

$$\frac{d^2m}{dx^2} - \frac{Et^3}{4CS} \cdot m + \frac{Et^3}{4CS} \cdot m_o = 0\,;$$

and the required differential equation becomes

$$\frac{d^2m}{dx^2} - am + b = 0 \qquad (5)$$

where

$$a = \frac{Et^3}{4CS}\,;\; b = \frac{Et^3}{4CS} \cdot m_o = am_o = \text{constant}$$

Solution of Eq. (5) will be:

$$m = A \sinh wx + B \cosh wx + \frac{b}{a} \qquad (6)$$

where $w = \sqrt{a}$ and A and B are constants determined from the boundary conditions of the beam ends being rigidly fixed, hence

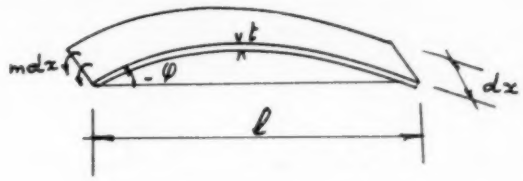

Fig. 3—Elementary strip of width dx and length l showing angle of rotation ϕ caused by torsion

$$A = 0\,; \quad B = -\frac{m_o}{\cosh \frac{wl}{2}}$$

Substituting for A and B in Eq. (6), the equation of the distribution of the resisting moment m of the inner slab becomes:

$$m = m_o\left[1 - \frac{\cosh wx}{\cosh \frac{wl}{2}}\right] \qquad (7)$$

Substituting for m from Eq. (7) in Eq. (1),

$$\frac{dT}{dx} = m_o \cdot \frac{\cosh wx}{\cosh \frac{wl}{2}}$$

Therefore, the equation of the distribution of the resisting torque of the beam will be:

$$T = m_o \cdot \frac{\sinh wx}{w \cosh \frac{wl}{2}} \qquad (8)$$

Special cases

Consider a beam of great torsional rigidity, i.e., $C = \infty$, hence $w = \sqrt{a} = \left[\frac{Et^3}{4CS}\right]^{\frac{1}{2}}$ is small.

Substituting for $x = \frac{1}{2}$ in Eq. (8) it becomes:

$$T = m_o \cdot \frac{\sinh \frac{wl}{2}}{w \cosh \frac{wl}{2}} = m_o \cdot \frac{\frac{wl}{2}}{w} = m_o \cdot \frac{1}{2}$$

This means that the total external moment m_o is resisted by the beam torque.

The other extreme is when $C = 0$, i.e., the beam will be just a knife edge support and $w \to \infty$. Substituting for $x = 0$ in Eq. (7) it becomes:

$$m = m_o\left[1 - \frac{\cosh w \cdot 0}{\cosh \frac{wl}{2}}\right] = m_o\left(1 - \frac{1}{\infty}\right)$$

$\therefore\ m = m_o$ which is the case of a slab with a cantilever part resting on an edge support.

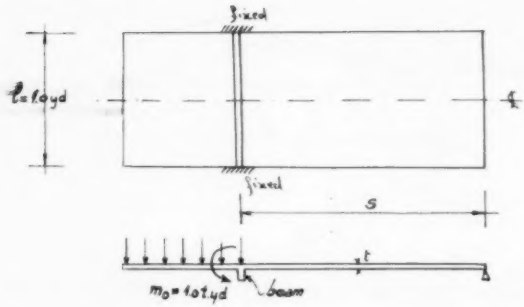

Fig. 4—Load on outer cantilever slab of unit width causes a torque on beam which supports inner slab of length S

Numerical example 1

Given an inner slab with a cantilever part supported on a beam of span 1 yd as shown in Fig. 4. The slab cantilever part will cause a bending moment —1.0 ton-yd. The inner slab and the beam are of such dimensions that $a = Et^3/4CS = (1 \text{ yd})^{-2}$

Draw the distribution of the inner slab moment and the beam torsion.

$$\text{Moment} \quad m_o = -1.0 \text{ ton-yd per yd}$$

$$\therefore b = m_o a = -1.0 \text{ ton per sq yd}$$

$$\text{and } w = \sqrt{a} = 1 \text{ yd}^{-1}$$

From Eq. (7) of the inner slab moment

$$m = m_o \left[1 - \frac{\cosh wx}{\cosh \frac{wl}{2}} \right]$$

$$= -1 \left[1 - \frac{\cosh x}{\cosh \frac{l}{2}} \right] = \frac{\cosh x}{1.128} - 1$$

and from Eq. (8) for the beam torque

$$T = m_o \cdot \frac{\sinh wx}{w \cosh \frac{wl}{2}} = -\frac{\sinh x}{1.128}$$

By substituting different values in the preceding equations, curves of the distribution of m and T are obtained as shown in Fig. 5.

Curves for slab moment m and beam torque T

Referring to Fig. 6 and 7, we have:

$$m = m_o \left[1 - \frac{\cosh wx}{\cosh \frac{wl}{2}} \right] \qquad (7)$$

$$T = m_o \left[\frac{\sinh wx}{w \cosh \frac{wl}{2}} \right] \qquad (8)$$

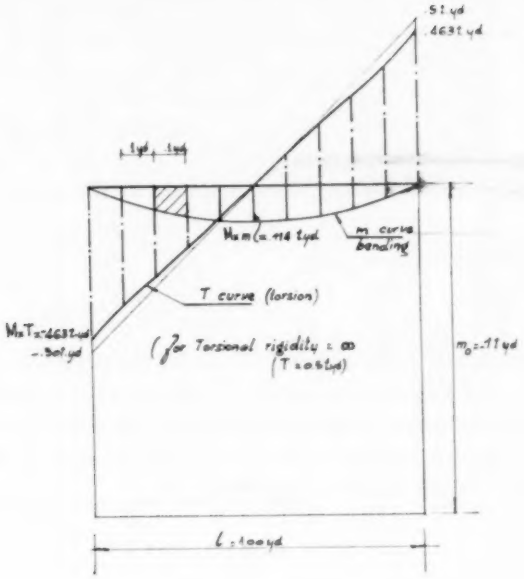

Fig. 5—Moment m and torque T distribution curves

Substituting for $u = wl$ in Eq. (7) and (8):

$$m = m_o \left[1 - \frac{\cosh \frac{ux}{l}}{\cosh \frac{u}{2}} \right] \qquad (9)$$

$$T = m_o \cdot \frac{l}{u} \cdot \frac{\sinh \frac{ux}{l}}{\cosh \frac{u}{2}} \qquad (10)$$

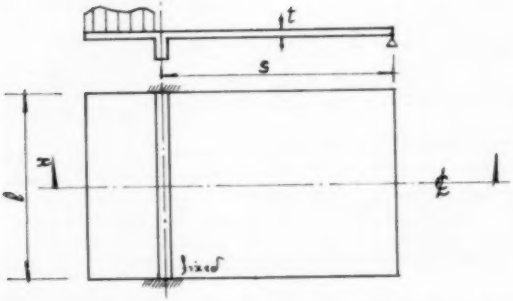

Fig. 6—Plan and section view of typical slab with its outer and inner sections supported by beam

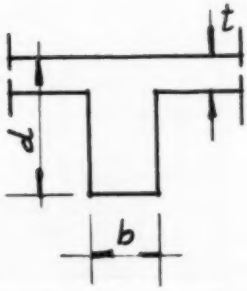

Fig. 7—Beam cross section

TABLE 2

$\frac{d}{b}$	1	1.5	1.75	2	2.5	3	4	6
k	2.035	141	124	1.13	0.97	0.86	0.72	0.57

where u is a coefficient denoted as the beam slab coefficient

$$u = \left(\frac{Et^3}{4CS}\right)^{1/4} l$$

$$\therefore u = \frac{K t l}{b^2}\left(\frac{t}{S}\right)^{1/4} \quad \text{(11)}$$

where K is determined from Table 2 for corresponding values of d/b:

For any case by substituting the values of t, l, b, S, and k in Eq. (11), the corresponding coefficient u is determined.

In the special case of slab thickness $t = 4$ in. and breadth of beam $b = 10$ in., Eq. (11) becomes

$$u = 0.08\frac{kl}{\sqrt{S}} \quad \text{(12)}$$

From Eq. (9), (10), and (11) curves of the distribution of slab bending moment m and beam torsion T are drawn as shown in Fig. 8 and 9 for different values of slab beam coefficient u.

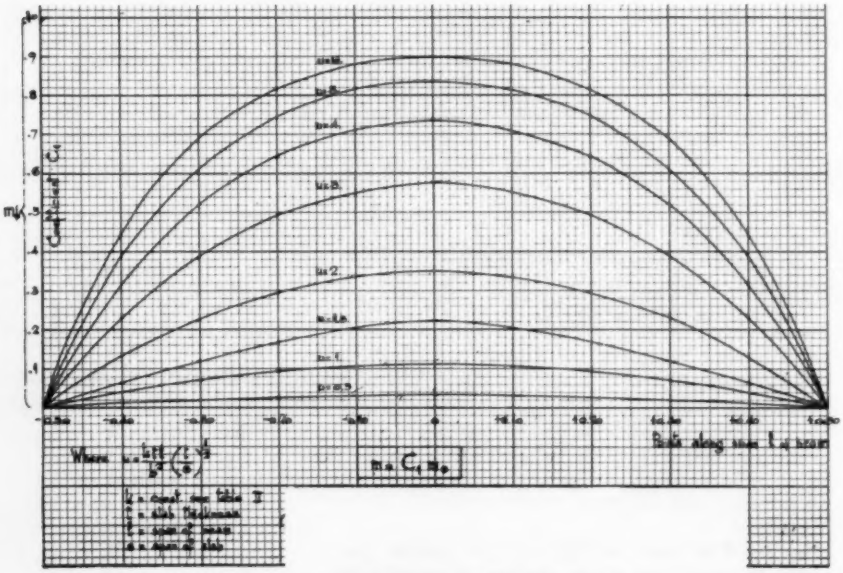

Fig. 8—Curves of moment taken by slabs

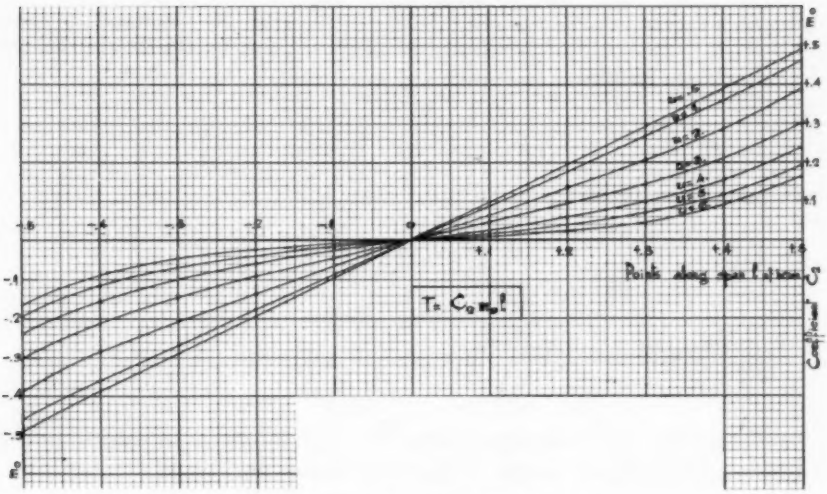

Fig. 9—Curves of torsion taken by beam

Numerical example 2

A beam 10 x 20 in. and of span $l = 255$ in. is subjected to a cantilever moment of 144×10^3 in.-lb on one side and supporting an inner slab of $t = 4$ in. and span 144 in. on the other side.

$$t = 4 \text{ in.}; \quad b = 10 \text{ in.}; \quad \frac{d}{b} = 2$$

$\therefore$ from Table 2 the corresponding value of $k = 1.13$

from Eq. (12) the slab beam coefficient

$$u = 0.08 \frac{Kl}{\sqrt{S}} = 0.08 \times 1.13 \times \frac{225}{\sqrt{144}} = 1.92$$

referring to the curves in Fig. 8 for $u = 1.92$ the coefficient $C_1 = 0.33$ at midspan of beam.

$$\therefore m = C_1 m_o = 0.33 \times 144 \times 10^3 = 475{,}000 \text{ in.-lb}$$

and from the curves of Fig. 9 for $u = 1.92$ the coefficient $C_2 = 0.4$ at end of beam,

$$\therefore T = C_2 m_o l = 0.4 \times 144 \times 10^3 \times 255 = 14.8 \times 10^6 \text{ in.-lb}$$

CASE II—MARGINAL BEAM

Marginal or spandrel beams are those beams at the outside wall of a structure or those framing around an opening. Such beams always receive load from one side only. Referring to Fig. 10, the slab ($l \times s$) is carrying a uniformly distributed load of g per unit area and connected to two beams where their ends are fully fixed.

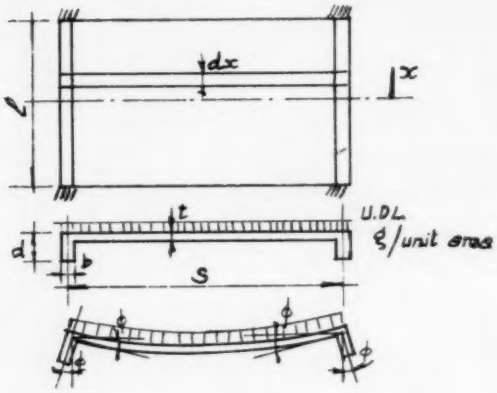

Fig. 10—Marginal or spandrel beams loaded on one side only

To determine the unknown forces m and T we shall consider the equilibrium of a short length of beam as shown in Fig. 11.

$$\frac{dT}{dx} = m$$

and since

$$T = C\, d\phi/dx$$

then

$$m = C\, d^2\phi/dx^2 \qquad (12)$$

By superposition and from Fig. 12, angle of rotation

$$\phi = \phi_1 + \phi_2 = -\frac{gS^2}{2Et^2} + \frac{6mS}{Et^3}$$

By substituting for $\frac{d^2\phi}{dx^2}$ in Eq. (12)

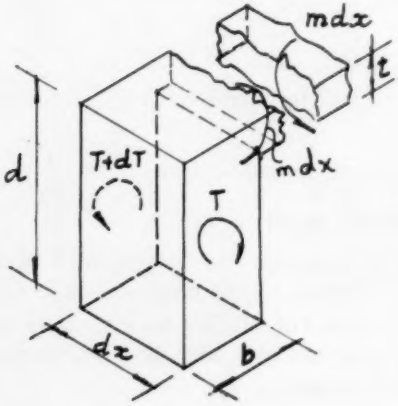

Fig. 11—Unknown forces, m and T, acting on short length of beam

$$m = \frac{6CS\dfrac{d^2m}{dx^2}}{Et^2}$$

and the required differential equation is

$$\frac{d^2m}{dx^2} - c^2m = 0 \qquad \text{where } c = \left[\frac{Et^3}{6CS}\right]^{1/2}$$

$$\therefore = A \cosh cx + B \sinh cx$$

from the boundary conditions $T = 0$

$$\therefore A = -\frac{gS^2}{12 \cosh \dfrac{cl}{2}} \qquad \text{and } B = 0;$$

Therefore, the equation for the distribution of the slab moment m becomes

$$m = -\frac{gS^2}{12 \cosh \dfrac{cl}{2}} \cdot \cosh cx \qquad (13)$$

but we have

$$\frac{dT}{dx} = m$$

and from symmetry, the equation for the distribution of the beam torsion will be:

$$T = \frac{gS^2c}{12 \cosh \dfrac{cl}{2}} \cdot \sinh cx \qquad (14)$$

Curves for m and T for the case of marginal beams

Similarly as for Case I by substituting for $\bar{u} = cl$ in Eq. (13) and (14) we have

$$m = -\frac{gS^2}{12 \cosh \dfrac{\bar{u}}{2}} \cdot \cosh \frac{\bar{u}x}{l} \qquad (15)$$

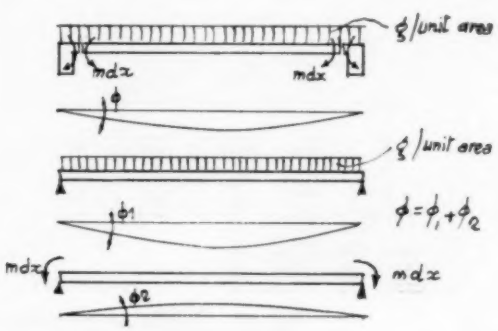

Fig. 12—Moment acting on marginal beam results in an angle of rotation ϕ

$$T = -\frac{gS^2\bar{u}}{12l\cosh\frac{\bar{u}}{2}} \cdot \sinh\frac{\bar{u}x}{l} \quad \text{.....(16)}$$

In this case the beam slab coefficient $\bar{u} = \bar{K}\frac{tl}{b^2}\left(\frac{t}{S}\right)^{1/2}$ where $\bar{K}$ is given in Table 3.

TABLE 3

$\frac{d}{b}$	1	1.5	1.75	2.0	2.5	3	4	6
$\bar{K}$	1.66	1.15	1.015	0.92	0.79	0.70	0.59	0.465

For the special case where slab thickness $t = 4$ in. and beam breadth $b = 10$ in.

$$\bar{u} = 0.08\frac{\bar{K}l}{\sqrt{S}}$$

From Eq. (15) and (16) curves for the distribution of slab bending moment m and beam torsion T are drawn as shown in Fig. 13 and 14 for different values of slab beam coefficient $\bar{u}$.

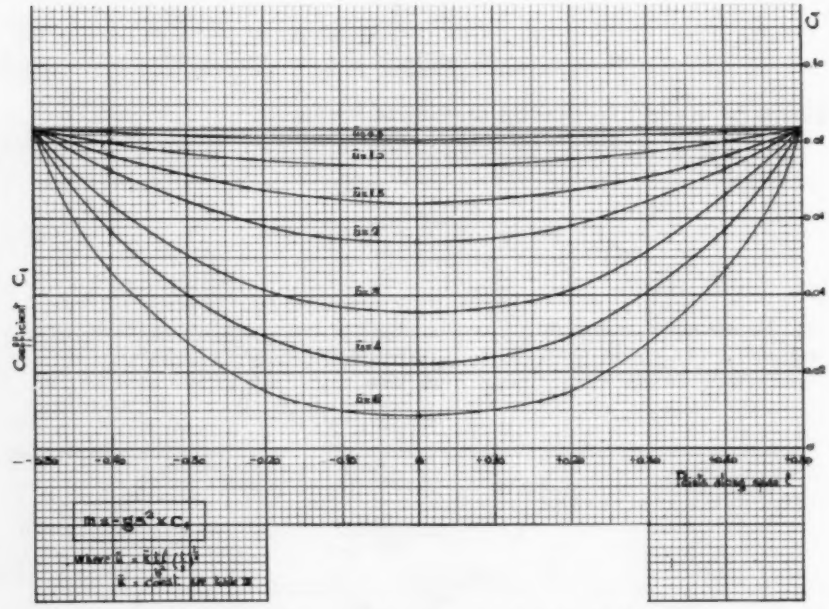

Fig. 13—Curves of moment taken by slabs

EFFECT OF BEAM END ROTATION ON THE DISTRIBUTION OF SLAB MOMENT m AND BEAM TORSION T

In the previous investigations the ends of the beam were assumed to be totally fixed, i.e., restrained against rotation. But it may happen in some cases that the end supports are not rigid enough and therefore will allow some rotation as shown in Fig. 15.

Ends of beam equally rotated

Let λ equal the angle of rotation of support for a unit couple acting at each end. Referring to Fig. 15, the angle of rotation at end of beam

$$\phi = T_{end}\,\lambda$$

and since

$$\phi = \frac{4S}{Et^2} \cdot m\,; \qquad T = C\,\frac{d\phi}{dx}$$

then the boundary conditions at the ends of the beam will be:

$$m = C\,\lambda\,\frac{dm}{dx} \qquad (17)$$

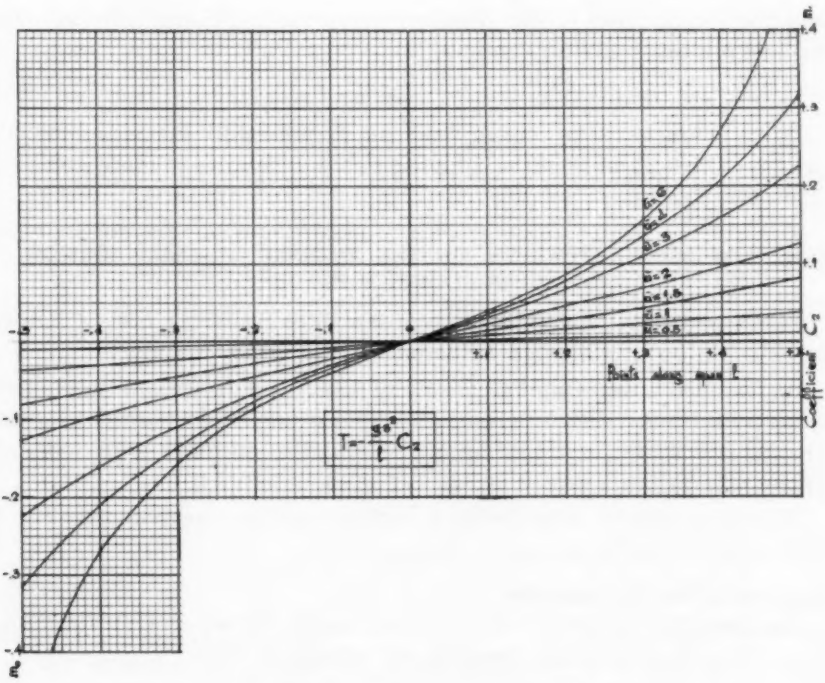

Fig. 14—Curves of torsion taken by beams

To get the effect of end rotation in Case I, Eq. (6) will have to satisfy previous boundary conditions which will give:

$$A = \frac{\alpha m_o}{(\alpha^2 - 1)\cosh\frac{wl}{2}}; \qquad B = \frac{A}{\alpha};$$

where $\alpha = C\lambda w$

The equation for the distribution of the resisting moment m of the inner slab taking into account the effect of the end rotation will be:

$$m = m_o\left[1 + \frac{\alpha}{(\alpha^2 - 1)\cosh\frac{wl}{2}} \cdot \sinh wx + \frac{1}{(\alpha^2 - 1)\cosh\frac{wl}{2}} \cdot \cosh wx\right]$$

putting

$$s = \frac{1}{(\alpha^2 - 1)\cosh\frac{wl}{2}}$$

then

$$m = m_o\left[1 + \alpha s \sinh wx + s\cosh wx\right] \qquad (18)$$

and the distribution of the resisting beam torsion taking into account the end rotation will be given by:

$$T = m_o\left[2x - \frac{s}{w}(\alpha\cosh wx + \sinh wx + \alpha)\right] \qquad (19)$$

In the case where the ends of the beam do not rotate the same amount, the procedure given may be adopted but will involve a more complex solution.

EFFECT OF BEAM DEFLECTION ON THE AMOUNT OF BEAM TORSION T AND SLAB MOMENT m

Effect of beam deflection is discussed by considering Case I. Other cases could be investigated by following the same procedure.

As for Case I, we assume the beam of span l is monolithically connected to a cantilever slab of width s carrying a uniformly distributed load of intensity W per unit area on one side and to an inner unloaded slab ($l \times S$) on the other side as shown in Fig. 16.

The beam will be subjected to a variable vertical reaction $r = Ws + m/S$ with its maximum value at midspan.

Required differential equation

Considering the equilibrium of a short length of the beam and denoting by y the deflection of the beam at any section X, then:

$$\frac{dT/dx}{c} = \frac{d^2\phi}{dx^2}$$

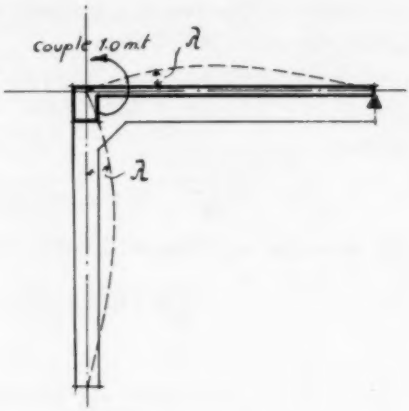

Fig. 15—Beam with both ends going through an equal angle of rotation (case of end supports not being rigidly held)

$$= \frac{d^2y/dx^2}{S} - \frac{4S}{Et^3}\frac{d^2m}{dx^2} \qquad (19a)$$

and by substituting for $\frac{dT}{dx} = m_o - m$ and $\frac{d^2y}{dx^2} = -\frac{M}{EI}$,

(where M is the bending moment of beam due to reaction r) then

$$\frac{d^2m}{dx^2} - am + \left(b - \frac{t^3}{4S^2I} \cdot M\right) = 0 \qquad (20)$$

where

$$a = \frac{Et^3}{4CS}; \qquad b = m_o a;$$

Comparing Eq. (5) and (20), it is seen they are both the same except for the constant b now decreased by the amount $\frac{t^3}{4S^2I} \cdot M$.

In most problems this value will not vary more than 30 percent of b, as will be shown later in the numerical example.

From Eq. (20)

$$\frac{d^2m}{dx^2} - am + a\left(m_o - \frac{C}{EI} \cdot \frac{M}{S}\right) = 0;$$

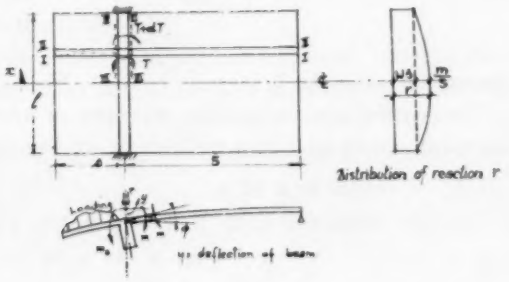

Fig. 16 — Beam deflection caused by loaded outer slab induces torsion and moment into inner slab

by assuming the bending moment $M = 0.1\ (ws + m/S)\ l^2$ and substituting for $m_o = Ws^2/2$ then

$$\frac{d^2m}{dx^2} - a\left(1 + \frac{0.1\,Cl^2}{S^2EI}\right)m + b\left(1 + \frac{0.2\,Cl^2}{sSEI}\right) = 0\,;$$

putting

$$\frac{0.1\,Cl^2}{SEI} = 0.5\,\frac{\beta}{S}\left(\frac{bl}{h}\right)^2 = z \quad \text{(for } \beta \text{ see Table 1)}$$

the required equation will be:

$$\frac{dm}{dx^2} - a\left(1 + \frac{z}{S}\right)m + b\left(1 + \frac{2z}{s}\right) = 0 \qquad \text{.....(21)}$$

$$\therefore\ m = A \sinh w_1 x + B \cosh w_1 x + \frac{b}{a}\cdot\frac{\left(1 + \frac{2z}{s}\right)}{\left(1 + \frac{z}{S}\right)}$$

where

$$w_1 = \sqrt{a\left(1 + \frac{z}{S}\right)}\,;$$

and

$$A = 0, \qquad B = -\frac{m_o}{\cosh\ \frac{w_1 l}{2}}\cdot\frac{1 + \frac{2z}{s}}{1 + \frac{z}{S}}.$$

The equation of the distribution of the resisting moment m of the inner slab taking into account the effect of beam deflection will be:

$$m = m_o \cdot \frac{1 + \frac{2z}{s}}{1 + \frac{z}{S}} \cdot \left(1 - \frac{\cosh w_1 x}{\cosh\ \frac{w_1 l}{2}}\right) \qquad \text{.....(22)}$$

and the equation for the distribution of the resisting beam torsion taking into account the beam deflection will be:

$$T = m_o \cdot \frac{1 + \frac{2z}{s}}{1 + \frac{z}{S}} \cdot \frac{\sinh w_1 x}{w_1 \cosh \frac{w_1 l}{2}} \qquad \text{.....(23)}$$

Numerical example 3

Determine maximum slab moment m and beam torsion T taking into account effect of beam deflection in Example 2.

beam 10 x 20 in. $\qquad l = 255$ in.

slab $t = 4$ in. $\qquad S = 144 \quad s = 48$ in.

$$m_o = 144 \times 10^3 \text{ in.-lb}$$

$$\frac{d}{b} = 2 \qquad \therefore \beta = 0.229 \qquad \therefore K = 1.13$$

From Eq. (12):

$$u = 0.08\,\frac{kl}{\sqrt{S}} = 1.92 = a \times l = a \times 255$$

$$Z = 0.5\,\frac{\beta}{S}\left[\frac{bl}{d}\right]^2 = 13$$

$$\therefore a = 56.5 \times 10^{-4}$$

$$\therefore\ w_1 = \sqrt{a\left(1 + \frac{Z}{S}\right)} = 0.00785$$

From Eq. (22)

$$m = m_o\,\frac{1 + \dfrac{2z}{s}}{1 + \dfrac{z}{S}}\left[1 - \frac{\cosh w_1 x}{\cosh w_1 \frac{1}{2}}\right]$$

$$m = 144{,}000\,\frac{1 + \dfrac{2 \times 13}{48}}{1 + \dfrac{13}{144}}\left[1 - \frac{1}{1.33}\right] = 710 \times 10^3 \text{ in.-lb}$$

Note in Example 2, $m = 475 \times 10^3$ in.-lb and from Eq. (23)

$$T = m_o\,\frac{1 + \dfrac{2z}{s}}{1 + \dfrac{z}{S}}\left[\frac{\sinh w_1 x}{w_1 \cosh w_1\, 1/2}\right]$$

$$T = 144{,}000 \times 1.41 \times \frac{1.175}{0.00785 \times 1.543} = 19.6 \times 10^6 \text{ in.-lb}$$

Note in Example 2 $T = 14.8 \times 10^6$ in.-lb.

CONCLUSIONS

1. Outer beams and slabs monolithically connected may be designed successfully by applying the derived formulas or curves.

2. When calculating the actual distribution of beam torsion or slab moment the effect of interaction of torsional rigidity between beam and slab is of great importance in the solution.

3. Economy in the reinforcement of the slab is realized and the connected beam must be reinforced to actual torsion moment especially at the ends.

4. Beam end rotation is important and depends on the stiffness of the supporting column.

5. Beam deflection will have the effect of increasing the calculated forces by 30 percent.

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Title No. 56-44

Shrinkage and Creep of Concrete

By INGE LYSE

In the design of prestressed concrete structures it is necessary to take into account the effects of shrinkage and creep, which cause a reduction of the initial prestressing forces. This paper reports on tests of the four major factors that contribute to shrinkage and creep. Equations are derived, based on the data obtained from the tests, for determining the combined shortening effects of creep and shrinkage.

■ ORDINARY REINFORCED CONCRETE as well as prestressed concrete structures are generally subjected to the effects of shrinkage and creep. In prestressed concrete structures the reduction in the prestressing force caused by shrinkage and creep may be of major importance. It is therefore necessary to provide for these time factors in the design of prestressed concrete structures. Certain countries have adopted empirical rules for the calculation of shrinkage and creep, but much more experimental data are still required before we may arrive at rational methods of calculating accurately their effects.

The major factors influencing the shrinkage and the creep of the concrete are:

1. The amount of cement paste in the concrete.
2. The physical and rheological properties of the materials in the concrete.
3. The temperature and relative humidity of the ambient atmosphere during curing and exposure.
4. The type and the intensity of the sustained stress in percentage of the ultimate strength of the concrete at the time of loading.

PROGRAM OF INVESTIGATION

Because of the limited time and loading rigs available, the investigation was concentrated on the study of only these major factors: the effect of the amount of cement paste in the concrete and the effect of the sustained stress in terms of the ultimate strength of the concrete at the time of loading. Two different concretes were used, the materials being ordinary French portland cement and Seine River sand and gravel,

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the one concrete having a water-cement ratio of 0.40 and a paste content of 33 percent for giving proper placeability with vibrators and the other having a water-cement ratio of 0.70 and a paste content of 27 percent. For the determination of the compressive strength of the concrete, 6-in. (15 cm) cubes were made, while for shrinkage and creep observations 2¾ x 2¾ x 11 in. (7 x 7 x 28 cm) prisms were used. All specimens were cured in the moist room at 68 F (20 C) and a relative humidity of 99 percent until the age of 7 days. The average 7-day compressive strength of the 6-in. cubes with $w/c = 0.40$ was 6250 psi (440 kg per sq cm) and of those with $w/c = 0.70$ was 3000 psi (210 kg per sq cm). For the creep tests the sustained stresses selected were: 20, 30, and 40 percent of the 7-day strengths of the two concretes.

SHRINKAGE AND CREEP TESTS

From each of the two concretes 12 prisms were made, six for shrinkage tests and six for creep tests. The prisms were removed from the moist room at 7 days and placed in a constant temperature room at 68 F and 50 percent relative humidity. For the creep tests two prisms of each concrete were placed in the individual loading rigs described in a recent paper by Mamillan,[1] shown in Fig. 1, and subjected to sustained stresses of 20, 30, and 40 percent of the strengths of the concretes at the time of loading. The observations were made with a fulcrum-plate strain gage having a 20 cm (8.0 in.) gage length.

The results of the shrinkage and the creep tests are shown in Fig. 2 and 3 for the two concretes. The sustained stresses were maintained for 115 days and then removed. The solid lines represent the average creep for the two companion specimens under the same sustained stress and the broken lines represent the average shrinkage of the six specimens of each concrete. It is noted that the creep curves show some irregularities at 115 days. This is due to the assumption that the modulus of elasticity of each concrete would remain constant during the time under load, while in fact both concretes would show some change in the modulus of elasticity due to the increased age. The shrinkage as well as the creep of the concrete of $w/c = 0.40$ and paste content of 33 percent was considerably greater than the shrinkage and the creep of the concrete of $w/c = 0.70$ and a paste content of 27 percent. As far as the shrinkage is concerned this is in agreement with earlier test results[2] where it was

found that the shrinkage of the concrete was approximately proportional to the percentage of cement paste in the concrete, regardless of the composition (water-cement ratio) of the paste. For a more direct study of the effect of the amount of cement paste in the concrete, Fig. 4 was prepared. Here the ratio between the shrinkages of the two concretes after various lengths of time in the room of 50 percent relative humidity is shown by the broken line. The ratios between the creeps of the two concretes are shown as solid lines. It is interesting to note that the creep ratios for 30 and 40 percent sustained stress follow the shrinkage ratio closely during the whole testing period, while the ratio for the 20 percent sustained stress is considerably higher, indicating an error in the stresses. Looking at Fig. 5 which shows the relationships between the creep and the sustained stress, it is noted that the stress at 20 percent for the concrete of $w/c = 0.70$ seems to be only about 83 percent of its proper value. Making correction for this as shown in Fig. 4 the creep ratio for the 20 percent sustained stress also follows the shrinkage ratio closely. The ratios are considerably above the paste ratio (which is $33/27 = 1.22$) for the first few weeks of tests, but they decrease gradually with the time and after a transition period of 8 to 10 weeks become practically constant at a value nearly equal to the paste ratio of 1.22. In other words the effect of the amount of cement paste in the concrete is just the same for the creep as for the shrinkage. *This means that when corrections are made for the amount of the cement paste in the concrete the creep of the concrete for a sustained stress equal to a certain percentage of the*

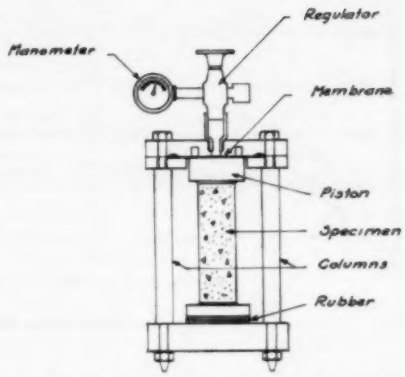


Fig. 1—Apparatus for creep tests

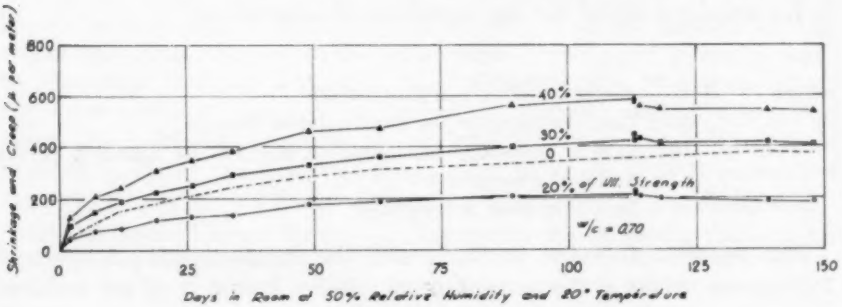


Fig. 2—Shrinkage and creep ($w/c = 0.40$)

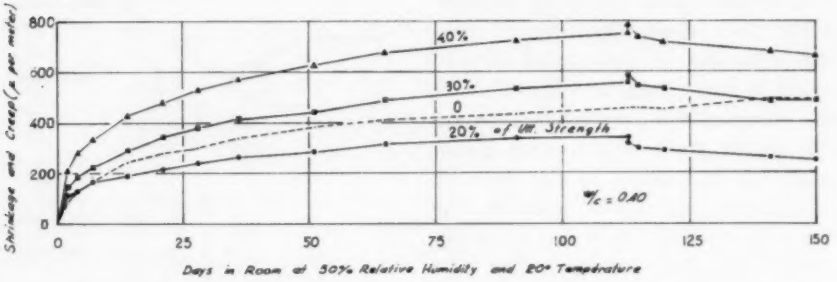


Fig. 3—Shrinkage and creep (w/c = 0.70)

strength of the concrete is of the same magnitude regardless of the water-cement ratio of the concrete.

The effect of the magnitude of the sustained stress on the creep of the concrete is shown in Fig. 5. All earlier tests show the same straight-line relationship between stress and creep as found here.[3,4] It seems that the results would give a creep of zero when the applied sustained stress was as low as 3 percent of the ultimate strength of the concrete at the time of loading. To clarify this point a few additional series of tests were made where the applied sustained stresses were only 5, 10, and 15 percent of the strength of the concrete. Although only short-time results are as yet available they indicate clearly that there is no load at which the creep is zero. The correct point of zero creep seems thus to be at a sustained stress equal to zero.

ANALYSIS OF RESULTS

In these tests as well as in all other investigations[3,4] the observed curves of shrinkage and creep have a common character of the kind shown in Fig. 2 and 3. Many investigators have proposed various formulas for these curves[3,4] and the simple one proposed by Aroutiounian in his excellent book[5] for the shrinkage of concrete is:

$$S(t) = S_o\,(1 - e^{-st}) \qquad (1)$$

where $S(t)$ = shrinkage after a drying period t
S_o = final shrinkage at the given relative humidity
s = rate of shrinkage depending on the relative humidity of the ambient atmosphere
t = length of time of shrinkage

This equation seems to fit fairly well the shrinkage curves obtained for various values of the percentage of relative humidity of the ambient atmosphere when the value of s is properly chosen.

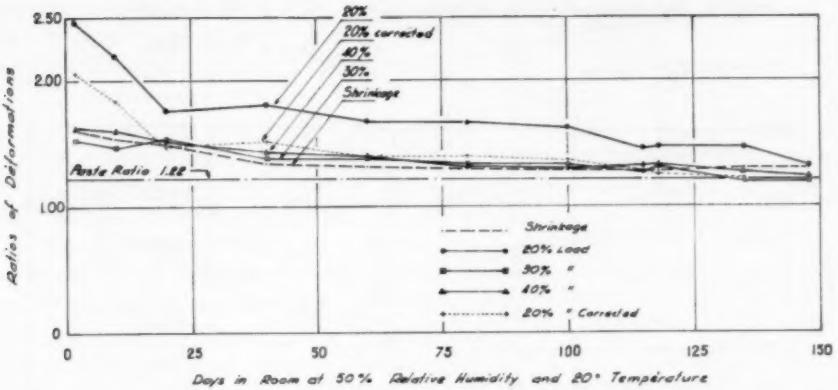


Fig. 4—Ratios of deformations of concretes having w/c of 0.40 and 0.70

For the creep curves Aroutiounian proposes a more complicated equation:

$$C(t,\tau) = \phi(t)\ [1 - e^{-\gamma(t-\tau)}] \quad \ldots\ldots (2)$$

As the present data as well as earlier investigations[2] have shown that the shrinkage of concrete containing a given cement and a given type of aggregate is approximately proportional to the paste content of the concrete regardless of the composition of the paste, we may write:

$$S(t) = S_{\infty}\ (1 - e^{-\gamma t})\ p \quad \ldots\ldots (3)$$

where: S_{∞} = final shrinkage per 1 percent paste in the concrete at the given relative humidity

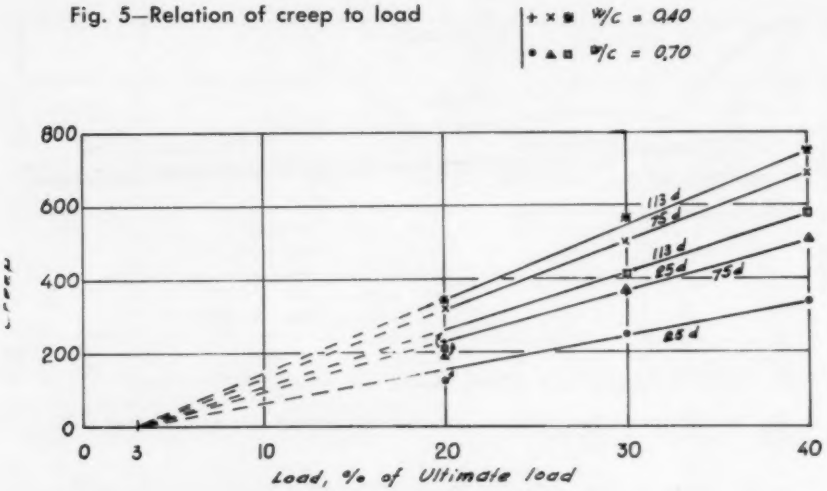
p = percent cement paste in the concrete

Considering the shrinkage and the creep curves shown in Fig. 2 and 3 we find that for 50 percent relative humidity of the ambient atmosphere a sustained stress of about 25 percent of the strength of the concrete will give a creep curve nearly equal to the shrinkage curve. For other percentages of relative humidity the magnitude of the sustained stress which will give creep equal to the shrinkage will be different. Results by Mamillan[1] indicate that for a relative humidity of 85 percent and a sustained stress of about 15 percent of the strength will give creep of about the same magnitude as the shrinkage, while at only 15 percent relative humidity the sustained stress must be as much as 35 percent of the strength for giving creep of the same magnitude as the shrinkage.

On the basis of these results we may set up the following formula for the creep of the concrete:

$$C(t) = S_{\infty}\ (1 - e^{-\gamma t})\ p\ \frac{\sigma_a}{\sigma_{ab}} \quad \ldots\ldots (4)$$

Fig. 5—Relation of creep to load



where: $C(t)$ = creep at time t
σ_a = actual sustained stress on the concrete
σ_{sh} = magnitude of sustained stress which will give creep equal to the shrinkage of the concrete at the given percentage of relative humidity

For practical design purposes the combined effect of shrinkage and creep must be considered. Combining Eq. (3) and (4) we obtain the following formula for the total shortening of the concrete:

$$Sh(t) = S_{oo}\,(1 - e^{-st})\left(1 + \frac{\sigma_a}{\sigma_{sh}}\right) d \qquad (5)$$

The unknown factors in the above equations are S_{oo}, s, and σ_{sh}. The value S_{oo} is obtained from the experimental curve for a time of observation t:

$$S_{oo} = \frac{S(t)}{(1 - e^{-st})\,p}$$

where the values for the various percentages of relative humidity of the ambient atmosphere must be observed for evaluating the factor s. Curves for such relative shrinkages are illustrated in Fig. 6. Such curves should be determined for the types of cements and aggregates used in the structure. Recent investigations of the creep of mortars of various cements[6] have shown that in general the creep is nearly the same for a sustained stress equal to a given percentage of the strengths of the mortars. Widely different aggregates may give considerable differences in the magnitude of the creep and the shrinkage. Creep and shrinkage curves for different aggregates should therefore be established.

Much concern has previously been given to the effect of the age at loading. If the sustained stress is taken in terms of definite percentages of the strength of the concrete at the time of loading, the age effect becomes of minor importance. As shown in earlier tests[2] the length of moist curing before placing in a drying room of constant relative humidity has only minor effect on the final shrinkage of the concrete. The size of the section has a definite effect on the shrinkage as well as on the creep of the concrete. For the application to full-sized structural members it is therefore necessary to investigate more thoroughly the relationship between the shrinkage and the creep of the relatively small specimens used in laboratory studies and of the actual full-sized sections in the structures.

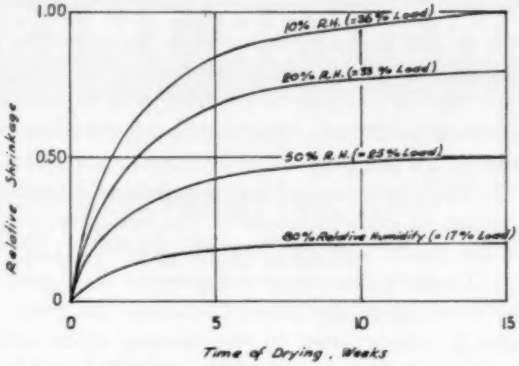


Fig. 6—Relative shrinkage at various humidities

To find the proper value of the sustained stress σ_{sh} which will give creep of equal magnitude as the shrinkage for the various percentages of relative humidity of the ambient atmosphere, a curve of the type shown in Fig. 7 should be established for the materials used in the structures. With accurately established curves of the kind shown in Fig. 6 and 7 it should be possible to calculate the effect of the shrinkage and the creep on ordinary reinforced concrete, and particularly on prestressed concrete structures, with a reasonable degree of accuracy.

For 100 percent relative humidity of the ambient atmosphere there is no shrinkage, in fact a small expansion, and the creep is also relatively small. Tests by Mamillan[1] indicated that the creep at 100 percent relative humidity was only about 40 percent of the creep at 50 percent relative humidity for sustained stress of equal percentage of the strength of the concrete at the time of loading.

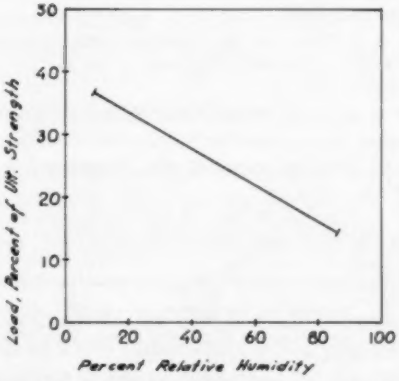


Fig. 7—Relationship between shrinkage and load for creep equal to shrinkage of concrete

CONCLUSIONS

It should be realized that only the major factors influencing the shrinkage and the creep of the concrete have been investigated here to obtain data readily adapted to the design of reinforced concrete and prestressed concrete structures. The following conclusions should therefore be considered for the purpose of practical applications only:

1. The shrinkage of the concrete is approximately proportional to the amount of cement paste in the concrete, regardless of the composition of the paste.

2. Under a sustained stress equal to a given percent of the strength of the concrete at the time of loading, the creep of the concrete is approximately proportional to the amount of cement paste in the concrete regardless of the quality of the concrete and the age at application of the load.

3. The relative humidity of the ambient atmosphere has a pronounced effect on the shrinkage and the creep of the concrete.

4. By means of Eq. (3), (4), and (5), the effects of the shrinkage and creep on reinforced concrete and prestressed concrete structures may readily be calculated with a fair degree of accuracy.

ACKNOWLEDGMENTS

The work reported was performed at the Laboratoires du Batiment et des Travaux Publics, Paris, and thanks are due R. L'Hermite, director of the laboratories.

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*Presented at the RILEM Symposium on Creep of Concrete, Munich, Nov. 1958.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

World-Wide Engineering and Scientific Publications

By ERNEST HARTFORD*

The critical needs of engineers and scientists for better access to the rapidly mounting volume of engineering and scientific publications are being studied by a science information service under the leadership of the National Science Foundation. James R. Killian, Jr., chairman of the President's Scientific Advisory Committee has said: "The increasing dimensions of world scientific publication has grown to the extent that it has become a problem requiring action at the national level." He said, "Science and engineering are largely built on the published record of earlier work done throughout the world. There are for example 55,000 journals appearing annually, containing about 1,200,000 articles of significance for some branch of research and engineering in the physical and life sciences. More than 60,000 different books are published annually in these fields, and approximately 100,000 research reports remain outside the normal channels of publication and cataloging. Within this vast body of world-wide scientific information, published and unpublished, lie the technical data that scientists and engineers need to do their work. The situation is further complicated by the fact that a large and important proportion of the world's scientific literature appears in languages unknown to the majority of American scientists and engineers —such as Russian and Japanese."

Nowadays engineering literature, like missiles, is directed on a world-wide basis. Furthermore the volume is such that, unlike conditions at the beginning of this century, every engineer needs an international index to keep abreast of the times. A good illustration proves the point. One of the founder engineering societies in 1911 held an annual meeting with a total of 14 papers—the same society at its 1958 annual meeting had 392 papers written by 656 authors, as some papers today are presented as a symposium, or prepared by two or more authors. Of the 392

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*Engineering Index, Inc., New York, N.Y.

papers only 165 will be published in full. Only 42 percent of the total papers presented before this important society meeting are made available to the engineering public. Only 298 were preprinted. Those preprinted are deposited in the Engineering Societies Library, and eventually are listed in the society's index of papers; but this index is not available until 18 months after some of the papers are presented, and then only those who consult that particular index may learn of their availability. Somewhat similar conditions exist regarding other national engineering societies.

Indexing and abstracting

The indexing of papers in the Engineering Societies Library is handled by Engineering Index, Inc., which is a nonprofit organization. Since 1885 the Engineering Index has provided indexing and abstracting service to engineers and other specialists who must keep informed of technological developments all over the world.

Engineering Index employs a staff of qualified editors who review over 1500 periodicals and society transactions, as well as a large number of bulletins and reports of government bureaus, schools, institutes, and research organizations. Publications in all branches of engineering and in all languages are reviewed. Articles which concern the application of engineering methods and concepts to any phase of the economy—and which are authoritative, informative, and useful—are abstracted.

The abstracts are easy to use. For each article reviewed, the following information is printed on a 3 x 5 library card: (a) subject heading classification, (b) title of article and name of author, (c) name and date of publication, and (d) brief summary of the article. Subscribers to individual divisions receive all pertinent abstracts once a week. Subscribers to the complete service receive their abstracts daily—about 30,000 a year.

All publications reviewed are permanently filed in the Engineering Societies Library. The library—which is open to the public—supplies, at minimum cost, photoprints, microfilm, and translations of the complete text of any article abstracted by Engineering Index.

Since the index covers all of engineering, its service is divided into 249 "field of interest" divisions. The subscriber who wants information in certain areas only, may take only the divisions he needs. The "field of interest" classification is for the convenience of subscribers in ordering particular divisions. For filing purposes, however, the literature is classified according to a standard list of subject headings. This makes it relatively simple to incorporate the abstracts into most reference files. The complete list of subject headings is supplied free to "complete service" subscribers and may be purchased by others—both subscribers and nonsubscribers.

The cost of the index varies according to the volume of abstracts prepared from the special field in which the subscriber is interested. The literature abstracted originates in 44 countries, and is published in 22 languages other than English. Time elapsing from the receipt of the publication until the abstract is distributed is reasonably brief.

Engineering Index cards subsequently appear as a bound volume cumulating all of the references of the year. This volume, now in its 75th year, is an internationally accepted digest of technological literature prepared for engineers, research workers, and students.

Bridges

New prestressed concrete bridge across the Danube-canal and the Freudenau harbor at Vienna (Die neue Spannbetonbrucke uber den Donaukanal und den Hafen Freudenau in Wien)

O. Mermon, *Osterreichische Ingenieur-Zeitschrift* (Vienna), V. 2, No. 2, Feb. 1959, pp. 39-56

Reviewed by H. H. Werner

Describes the planning, layout, Dywidag prestressing system, and free-cantilever construction method used for this bridge.

Highway bridge foundation practice in Queensland

I. G. Cameron, *Journal*, Institution of Engineers, Australia (Sydney), V. 31, No. 7-8, July-Aug. 1959, pp. 163-175

Author's Summary

Deals with the design and construction of foundations for short and medium span highway bridges. The structural and economic considerations which enter into the choice of foundation type are discussed. Queensland streams differ from those in most other parts of the world in the great heights attained by the streams during flood. Flood heights of 80-90 ft. above normal water level have been recorded in a number of streams, and 45-50 ft flood rises are commonplace. Queensland streams also carry large quantities of debris which can cause severe transverse loading on the bridge. It is therefore essential to design the majority of bridges as submersible structures. Design criteria and procedures for this type of structure are described. Methods used by the Queensland Main Roads Department in the construction of bridge foundations are also described in some detail.

Deck holes plague bridge, but consultants zero in on the trouble

Engineering News-Record, V. 163, No. 21, Nov. 19, 1959, pp. 48-49

This article points up the results of studies made to determine the cause of deterioration and the corrective measures for repairs on the deck of an 8 year old bridge on the New Jersey Turnpike. The cost of repairs has been more than 60 percent of the initial cost of the deck. The findings are interesting.

Cores tested at an average of only 2500 psi. (Cylinders taken during placement tested at 3000 psi.) The concrete is more porous and less dense than anticipated. There are readily discernible variations in the appearance and texture of the concrete at different locations. In many places the reinforc-

Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

ing steel is closer to the surface than it should be. The cement content of all samples tested was greater than required by the specifications. Stone and sand met the requirements of the specifications. No correlation is evident between variations in surface texture or appearance and the incidence of failure.

The earliest and most severe failures occurred at points where the bars were close to the surface (as little as ½ in. cover in places). The porosity of the concrete is blamed as a principal source of trouble. Almost invariably, slabs that were cast on warm, dry, windy days showed checked surfaces indicating that proper measures for hot weather concreting were not taken.

Corrective measures being taken in the repairs include increasing the slab thickness, increasing the protective cover for the reinforcement, fixing the position of the reinforcement, using extreme care in concrete control, improving methods for batch control, and taking special precautions to prevent rapid drying.

Circular reinforced concrete columns subjected to direct stress and bending

A. ANTHONY TOPRAC, *Public Roads*, V. 30, No. 11, Dec. 1959, pp. 241-245

AUTHOR'S SUMMARY

Develops a procedure for designing eccentrically loaded round columns based on the American Association of State Highway Officials' bridge specifications for allowable loads.

The principal benefit of this article will be derived from the method outlined in the text, which will assist bridge designers who may want to expand the graphs presented or make similar graphs for different bridge column sections. Plotted curves are given for a range of 24- to 60-in. diameter columns in 6-in. increments and for three quantities of reinforcing steel. In design problems for which the graphs are applicable, considerable tedious work will be avoided.

Designing small span bridges

HAROLD F. SOMMERSCHIELD, *Consulting Engineer*, V. 12, No. 2, Feb. 1959, pp. 102-105

Reviewed by ARON L. MIRSKY

Reports design and construction methods used for four small (nominal 40 ft) span prestressed bridges in the Chicago area, to obtain a total deck cost of $8.00 per sq ft. Problem of reducing top tension in concrete was met by encasing (shielding) some of the strands in paper tubes near the supports, thus meeting all requirements for use of thin box girders.

Clifton Bridge, Nottingham: Initial design studies and model tests

ROBERT M. FINCH and A. GOLDSTEIN, *Proceedings*, Institution of Civil Engineers (London), V. 12 (session 1958-59), Mar. 1959, pp. 289-316

Clifton Bridge, Nottingham: Design and construction

ROBERT M. FINCH and A. GOLDSTEIN, *Proceedings*, Institution of Civil Engineers (London), V. 12 (session 1958-59), Mar. 1959, pp. 317-352

Reviewed by ARON L. MIRSKY

Two interesting papers on an interesting structure.* Bridge, of prestressed concrete construction, crosses the River Trent on a 24 deg skew. The main span, 275 ft long, is composed of two 87.5 ft cantilevers and a 100 ft suspended span; there are also three 90 ft viaduct spans and two 125 ft end spans, for an over-all length of 800 ft. Design problems included provision for floods, navigation, subsidence due to mining, utilities, nonsymmetry of deck (future construction will convert bridge to a dual-highway type, hence present structure provides for a 22 ft roadway, a 9 ft cycle track, and a 6 ft sidewalk), and the skew. Bridge was originally intended to be square, and was later made skew; design was thus on basis of a square structure, with modifications due to skewing (authors conclude that a skew bridge of this nature should be avoided if possible and a square crossing substituted, but they make no

*cf, *The Engineer* (London), V. 203, No. 5289, June 7, 1957, pp. 872-873; "Current Reviews," ACI JOURNAL, V. 29, No. 7, Jan. 1958 (*Proceedings* V. 54), p. 623

mention of what the traffic engineers' reactions would be.) Aesthetics of structure were passed on by the Royal Fine Art Commission. Vibration characteristics of the slender structure were also investigated.

Model tests were primarily to check analytically-obtained values of deflections due to load and prestress and the effects of the skew. Correlations were good. Experimental setup is an excellent example of what can be accomplished, given ingenuity, thought, and authority.

Second paper covers substructure and superstructure design, with many sketches of details such as joints, and construction. Interestingly enough, the main-span cantilevers were constructed by the free-cantilever method.

Construction

Floor and roof systems

Building Construction Illustrated, V. 28, No. 5, Nov. 1959, pp. 30-55, 61

A capsulized compilation of floor and roof systems currently available, covering such types as cast-in-place concrete, precast concrete joists, precast-prestressed concrete, barrel vault lift slab, composite concrete and steel, concrete joist-filler tile, shoring and forming, and costs.

The Phoenix - Rheinrohr skyscraper, Dusseldorf (Hochhaus Phoenix-Rheinrohr, Dusseldorf)

Die Bauzeitung (Stuttgart), V. 64, No. 5, May 1959, pp. 216 ff (also p. 192)
Reviewed by Aron L. Mirsky

This skyscraper, one of the few examples of this type of structure so far built in Germany, is 84.50 x 22.94 m in plan and 105.32 m in over-all height (94.66 m above street level). Above ground, structure has a steel skeleton (floors are composite type); below ground level, the three-story foundation block is of reinforced concrete, and there is an underground garage, also of reinforced concrete, which is structurally independent of the skyscraper foundation; the two independent structures are separated by an 80 cm joint to accommodate possible differential settlement. Because of the high ground-water table, the foundation blocks are encased in a waterproof "tub" of aluminum foil. Also of considerable interest is the design and detailing of the reinforcement in the column footings to take high wind-load uplift forces.

Building is one of many new and interesting structures being built as part of "urban redevelopment" of Dusseldorf, which is the theme of this issue of *Die Bauzeitung*.

Ordinary concrete meets tough spex

Contractors and Engineers, V. 56, No. 8, Aug. 1959, pp. 28-29

Discusses the use of ordinary concrete in the construction of a high-level radioisotope test cell in the Picker Research Center. This test cell is 6 ft square with 5½ ft thick walls and 4 ft thick floor and ceiling and required 300 cu yd of concrete which weighed more than 600 tons. Although high-density concrete and concrete blocks were considered, they were not employed because of cost, density, uniformity, etc.

Three trial mixes were used before such problems as overheating, thermal cracking, shrinkage, and early set were eliminated. The final mix has a 28 day average strength of 5736 psi and weighed 155.4 lb per cu ft.

Earthquakes—Construction inspection

Earthquake Engineering Research Institute, San Francisco, 1959, 24 pp.

This manual is intended to serve as a guide for inspectors and building officials in the field inspection of the construction of buildings and structures which must resist the effects of earthquakes and which must be safe for occupants where earthquakes occur. The manual is applicable for those areas and communities which have provisions for earthquake safety in

their building codes. The manual covers inspection of the folowing structural materials used in earthquake resistant construction: brick masonry, concrete and clay tile, hollow block masonry, structural gypsum, structural wood, reinforced concrete, structural and light gage steel, foundation beds.

TVA builds a floating guard wall for new Wilson Lock

Engineering News-Record, V. 163, No. 14, Oct. 1, 1959, pp. 40-42, 44

TVA is constructing a 553 ft long floating guard wall or boom as part of a new 100 ft long navigational lock. This lock, in turn, will be part of the new Wilson Dam on the Tennessee River. The boom is made of concrete and consists of six pontoons rigidly bolted together. The deck is 27 ft wide and 13 ft deep and the end will be anchored by using deadmen and securing cables.

Construction of a large experiment tank at Haslar

JOHN W. HUNTER, ARTHUR C. LITTLE, and JOHN D. HACON, *Proceedings*, Institution of Civil Engineers (London), V. 12 (Session 1958-59), Jan. 1959 pp. 57-82

Reviewed by ARON L. MIRSKY

Built for the Admiralty, this covered concrete tank 400 x 200 x 18 ft is designed to permit testing of ship models under simulated conditions of calm sea or waves. The site is just above high tide level; this and the foundation material, for which large settlements were predicted, led to the use of a "flexible" structure on piles, e.g., 20 ft high walls act both as cantilevers and as slabs spanning between buttresses. Roof framing utilized welded steel trusses of tubular material; details of these are also given.

How close planning pays off on 2000-pupil high school project

Building Construction Illustrated, V. 28, No. 3, Sept. 1959, pp. 38-41

Reviews the construction and design of a high school library. The architect desired a concrete dome roof, covering an area 80 ft in diameter, with an interesting shape. To obtain this end the engineer designed a roof consisting of 16 pie-shaped segments tied into ribs, each segment a portion of a cylinder. The paper further describes the contractor's method of supplying the intricate formwork needed, and the problem encountered in placing the vast maze of reinforcing steel.

Construction Techniques

Barrel vault lift slab at $2.70 psf

Building Construction Illustrated, V. 28, No. 3, Sept. 1959, pp. 44-45

Describes in brief the erection of a five barrel vault roof, formed on the ground and then raised to its 13 ft height by hydraulic jacks. Each barrel is 85 ft long by 24 ft 4 in. wide and has a horizontal steel tie rod connecting the edges which compensates for the horizontal thrust thus enabling the structure to act as a flat slab. The total cost of the roof was $2.70 per sq ft.

Investigation of methods of preparing horizontal construction joints in concrete

W. O. TYNES, *Technical Report* No. 6-518, U. S. Army Engineer Waterways Experiment Station, July 1959, 28 pp., $0.50

AUTHOR'S SUMMARY

A series of strength (shear, tensile, and flexural) and permeability tests were made using simulated interior mass concrete to evaluate methods of preparing horizontal construction joints. The results indicated that: thorough cleaning of the lower surface was essential to obtaining a strong joint; keeping surfaces wet for several hours prior to placing the new concrete appeared to decrease joint strength; placing mortar on the lower surface immediately before the fresh concrete was placed rather than placing concrete on concrete appeared to decrease joint strength; leaving the texture of the lower surface rough so as to obtain the benefit of the mechanical key-

ing action of the larger aggregate particles did not improve joint strength; no significant difference was noted in quality of the joints because of aggregate size; cleaning surfaces that had been treated with a retarder by water jet allowed for formation of joints superior to those formed by surfaces that had not been cleaned. The incomplete removal of the retarded layer of concrete did not impair the integrity of the joint later made against it.

Concrete wales brace circular cofferdam

Engineering News-Record, V. 163, No. 15, Oct. 8, 1959, pp. 36-38

A circular cofferdam had to provide an inside clear space 101 ft in diameter to contain the structure housing the equipment for a new sewage pumping station. It was 64 ft from the top of the ground to the bottom of the reinforced concrete structure. The contractor decided to use cast-in-place reinforced concrete wales to brace the cofferdam. Steel sheet piling was driven next to an existing reinforced concrete sewer that could not be disturbed. Then 65 ft long H-beam soldier beams were driven on 7-ft centers, around 101 ft diameter circle. With a clamshell excavating inside this circle, timber lagging was sawed to 6.5 ft lengths and inserted against the face of the cut. As the hole got deeper, the thickness of the lagging increased. As the excavation attained the desired depths, the concrete wales, 3 ft wide by 2 ft thick, were cast in place on the ground at that level. Each ring contains about 75 cu yd of concrete, is reinforced with four #6 bars in both the 2 ft deep inside and outside faces and #4 bar stirrups at 6 ft center to center. The wales are held in position vertically by heavy reinforcing bars at a 45 deg angle, welded to alternate soldier beams. Felt was placed between the circular concrete wales and the outer flange of the soldier beams so that the soldier beams could be removed after backfilling around the reinforced concrete pumping station structure had been completed. Wales were at depths of 17, 27, 36, 39, 50, and 55 ft below ground elevation. The article also describes other construction procedures including method of cutting into the existing sewer.

King-sized steel pans simplify deck forming

Contractors and Engineers, V. 56, No. 8 Aug. 1959, pp. 90-93

Reports briefly on the use of 38 ft long deck pans for constructing a bridge on the Austin-San Antonio expressway. These pans not only eliminate all falsework, as they rest on timbers bolted to the pier caps, but also greatly reduce time and cost of construction.

Recent experience with the powder lance

Concrete Construction, V. 4, No. 7, July 1959, pp. 12, 13

Reports briefly on the use of a powder lance employed to cut up concrete structures to facilitate their removal. This lance, burning a mixture of aluminum and iron powder in contact with pure oxygen, literally melts the concrete with a 6000 deg flame.

Welding rebars on winter job saves money

Bill Allen, *Contractors and Engineers*, V. 56, No. 10, Oct. 1959, pp. 68-70

Describes protective measures taken for winter concreting of building foundation slabs 4 ft 9 in. thick for a group of cement silos. In places, the heavy #11 bars were crowded into the forms in five separate layers. The bars, often spaced less than 2 in. apart, had to be positioned accurately. Welding of the rods increased production enough to justify the additional cost of the welding rod and the operation of two welding machines.

Before the concrete was placed, the wood forms were covered with tarps, and space heaters warmed up the steel

and forms to well above the freezing point. Preheated ready-mixed concrete, with a 1 percent calcium chloride additive, was then bucketed to the forms. Shortly after the concrete was finished, glass fiber insulated concrete curing blankets were laid on the surface. While the blankets were in place, the temperature of the surface of the concrete never dropped below 75 F. Normally, they were left on the concrete for about 5 to 7 days. The insulation blankets stood the wear-and-tear of the construction job quite well and were re-used about ten times.

Flame cuts slab in 1/3 time

Construction Equipment, V. 20, No. 3, Nov. 1959, p. 58

Describes the removal of a reinforced concrete slab, 325 ft long, 2 ft thick, and 15 ft wide, by cutting it into slices with a power-lance. The intense heat of the power-lance is produced when a special mixture of iron and aluminum are combined with oxygen. The entire removal took 8 days which was only ⅓ of the time required by ordinary methods.

Concrete—An art or a science?

L. Boyd Mercer, *Modern Concrete*, V. 23, No. 7, Nov. 1959, pp. 38-41

A controversial article, the author's views are based on the conclusion that the number of variables involved in concrete is too great for the effective application of mathematics to this construction material. He outlines the weaknesses of the cement-water ratio and accepts the opinion that no precise mathematical relationship exists between the ratio of water to cement and the compression strength due to the many and frequently obscure variables of cement manufacture.

Inasmuch as an infinite number of aggregate gradings can have the same fineness modulus index, he feels that mix proportioning by the use of the fineness modulus cannot provide a

With regards to concrete and the materials used in the batching, mixing, and testing, conditions are so excessively and irregularly variable that the accepted mathematical procedures for statistical control can have little direct application. He feels that tests mean only that the material in the structure should be capable of withstanding similar compressive stresses if compacted, cured, and tested in an identical manner and the safety factor must provide for the basic variable of cement quality, batching errors, mixing conditions and testing variables, variations in the placing of reinforcement, and errors in assumptions and theory of design procedure.

Finally specifications must be realistic and there must be complete cooperation and confidence between the specification writers and users. The author concludes that concrete production is an art and should be based on previous experience with adjustments made for variations in materials and conditions.

Building winter resistance into concrete

Fred F. Bartel, *Concrete Construction*, V. 4, No. 11, Nov. 1959, pp. 7-8

A summary of good practice to follow to improve resistance of concrete to severe weathering and to deicer salts.

Form contraction joints with plastic strips

Bill Allen, *Contractors and Engineers*, V. 56, No. 8, Aug. 1959, pp. 40-41

Discusses the use of plastic strips for forming transverse contraction joints in the runways of a new Navy air base. These plastic strips which are placed in the fresh concrete by machine one day and removed by hand the next day give smooth walled joints which are then filled with sealing compound. They are easily extracted from the concrete because of a removable V-shaped insert which allows the walls of the strip to collapse. They also need no

lubricating oil or coating of any kind and may be re-used an average of 25 times.

Sculpting with sprayed concrete

Architectural Forum, V. 111, No. 4, Oct. 1959, pp. 167-168

Brief description of "shelter sculpture." Use of shotcrete on shaped reinforcement backed with paper-backed mesh to form unique house and exhibit pavilion. One completed example is pictured.

Continuously moving framework in the construction of an 8000 ton capacity grain storage silo

D. M. Magee, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 84, June 1957-May 1958, pp. 13-48

Continuously moving slip forms were used in the construction of an 8000 ton capacity grain import silo. Because silo had to be close to the waterway and there was no land available, the silo was built on a pile foundation in the river bed. The essential features of a moving formwork construction are discussed: wall forms, yokes, working deck, scaffolding, climbing bars, and jacks. In discussing the method of operation, author explains jacking rate, concreting, steel fixing, leveling of the forms, roof construction, and striking of panels. Summarizes general considerations and discusses apparent advantages of this system. Gives example for practical determination of a value for friction.

Concreting problems and methods in hot climates

D. P. Harris, *The Reinforced Concrete Review* (London), V. 4, No. 2, June 1956, pp. 95-106 and pp. 115-125; discussion pp. 106-114
Reviewed by William R. Lorman

Emphasizes the importance of correct management (from the highest to the lowest levels) and high standards if concrete construction in the tropics is to be accomplished properly.

Though short, this contribution elucidates on many practical factors incident to making concrete in tropical areas such as western and eastern Africa and southern Burma. Included are personnel problems, characteristics of cement and aggregates, and climatic effects on construction methods. Four tables and 22 photographic illustrations (pp. 115-125) supplement the information presented in the text. The discussion reflects the practical views of nine experts.

Dams

Design of Catagunya prestressed dam

J. K. Wilkins and J. Fidler, *Civil Engineering Transactions*, Institution of Engineers, Australia (Sidney), V. CE 1, No. 2, Sept. 1959, pp. 47-64 (including discussion)
Authors' Summary

Describes the factors leading to a decision to construct a prestressed concrete dam 150 ft high on the River Derwent in Tasmania. Problems encountered in the dissipation of the energy of flood waters are discussed. Details are given of the design criteria and stresses adopted and of model experiments carried out to assist in determining the resistance of the foundation rock to pull out of the cables. The economies that can be achieved by prestressing are discussed and a rapid method of estimating the optimum distribution of prestressing steel is given. The dam when completed will be the highest prestressed dam in the world designed from its inception on the Coyne system. Suggestions are made for further research.

Design

Frames and arches

Valerian Leontovich, McGraw-Hill Book Co., Inc., New York, 1959, 472 pp., $20

For the first time a solution for analysis of frames with members of variable cross section, based on the classical method, is presented in a convenient practical form. All computations with this method are confined to simple, short operations; the time required for frame analysis is greatly

reduced. In addition to giving a new solution of frames with members of constant cross section, the book also provides comprehensive tables and graphs for a wide range of straight and curved members of variable cross section. In persenting 400 condensed solutions for 20 principal types of statically indeterminate frames and arches, the book furnishes a complete and invaluable professional tool for solution of frames usually encountered in the engineering practice. Some of the objects of this book are to provide a convenient scientific tool for the analysis of arches and frames with members of variable cross section; to present these results in the form of equations which can be solved by a slide rule, still yielding sufficient accuracy in the result; and to demonstrate the inherent precision and brevity of the elastic center method.

The first part of the book covers arches and frames with members of constant cross section. The second part is devoted to similar structures with members of variable cross section. Expressions for redundant quantities as well as the bending moment and shearing in axial forces at any section of the structure are provided in the text for many vertical and horizontal loading conditions. Solutions for complex structures are presented in such a manner that only algebraic computations are required. In the appendix, numerical values of elastic parameters and load constants for straight and curved members of variable cross section are provided in charts and tables.

Analysis of curved girders

H. H. Fickel, *Proceedings*, ASCE, V. 85, ST 7, Sept. 1959, pp. 113-141

Author's Summary

Paper presents the derivation of influence lines for bending and torsional moments and shears on horizontally curved girders. Solutions are given for simple and continuous spans. Diagrams are supplied showing the characteristic values of curved girders with constant moments of inertia. The effects of variable moments of inertia and also of the elasticity of the torsional supports are discussed.

Load distribution in concrete joist floors with cross-ribs (Zur Lastverteilung bei Rippendecken mit Querrippen)

H. Meischl, *Osterreichische Ingenieur-Zeitschrift* (Vienna), V. 2, No. 8, Aug. 1959, pp. 291-294

Reviewed by H. H. Werner

Analysis of load transfer of cross-ribs to adjacent joists. The general conclusion is that cross-ribs distribute concentrated loads fairly evenly over five joists.

Analysis of concrete slabs on ground

G. A. Leonards and M. E. Harr, *Proceedings*, ASCE, V. 85, SM3, June 1959, pp. 35-58

Highway Research Abstracts Sept. 1959

A theory has been developed whereby the stresses, deflections, and degree of support can be computed for symmetrically loaded circular slabs subject to warping caused by ambient temperature and/or moisture gradients.

Comparison with available theories for fully supported slabs indicates that, for conventional slab sizes, the shape of the slab has only a minor influence on the critical stresses.

The maximum equivalent temperature differences between slab surfaces at which a warped slab can maintain full contact with its suport are much lower than those frequently encountered in practice.

Numerical solutions for the case of an unloaded slab (apart from its weight) with a free edge boundary were obtained with the aid of a digital computer. Comparisons with available measurements show, that for this case, the theory can reliably predict the deflections even if warping is sufficiently severe to result in a partially supported slab.

The theory shows considerable promise as a useful tool for the analy-

sis of a number of problems concerning the behavior of concrete slabs on ground. Its general validity can be appraised by means of experiments in which temperature and moisture gradients are carefully controlled, and reliable, simultaneous measurements of the deflections, strains, and degree of support are obtained with reference to the initial unwarped condition.

Structural frame analysis by electronic computer

E. Lightfoot and F. Sawko, *Engineering* (London), V. 187, No. 4843, Jan. 2, 1959, pp. 18-20
Reviewed by Aron L. Mirsky

Generalized slope-deflection is applied to the problem of grid frameworks, using electronic computers. Master program for computer is based on fact analyses for plane and grid frameworks are parallel, by this method, and the basic generalized slope-deflection equations are similar in the two cases. Torsion, members with different properties, and similar factors are easily accommodated in the program. Results of two numerical examples are given, the first being checked against A. W. Hendry's experimental (model) investigations (Hendry, A. W., and Jaeger, L. G., *Journal*, Institution of Civil Engineers, Dec. 1955, pp. 939-971; "Current Reviews", ACI Journal, V. 28, No. 3, Sept. 1956 (*Proceedings* V. 53), p. 327) and the second, representing a theoretical problem, being checked out statically.

Seismic forces in buildings and structures (in Russian)

I. L. Korchinski, Scientific and Technical Society of Construction in the U.S.S.R. (Moscow), 1959, 77 pp.
Reviewed by E. Rosenblueth

This manual is intended to justify new Russian provisions for aseismic design and explain their application through several examples. New provisions call for dynamic analysis. Design is based on envelope of natural modes, each scaled in accordance with a base shear coefficient that corresponds to an acceleration spectrum which gives a constant ordinate for natural periods up to 0.3 sec and a hyperbola from there on, with a constant cut-off at a natural period of 1.5 sec. For masonry buildings whose dynamic analysis is not easy, it is licit to assume a triangular distribution of horizontal accelerations, and a table of shear coefficients is given for each story in terms of the number of stories. Correction factors are specified for structures having low energy absorption capacity. The manual includes approximate formulas and charts for numerous structures, such as chimneys and silos, which in some cases take into account foundation compliance.

Derivation of the design spectrum is debatable, for it is assumed that each natural period responds to a single damped sine wave. However, adequacy of the design method is substantiated by analysis of more than a dozen structures, most of them damaged by earthquakes, whose behavior checks well with the criteria specified. Manual includes a list of both Russian and Western references.

Tie reinforcement (Estribos para armar)

E. Rosenblueth and J. Laborde, *Revista Ingeneria* (Mexico), V. 29, No. 2, Apr. 1959, pp. 5-13
Authors' Summary

Authors propose limitations for spacing of stirrups and ties in beams, girders, and columns where this reinforcement is not required for diagonal tension. In beams and girders the spacing is governed by considerations of buckling or excessive deflection of the stirrups due to load applied while placing concrete, excessive deflection of longitudinal reinforcement in the top layer, and vertical shrinkage reinforcement requirements. The loads assumed are based on measurements and observations on numerous jobs. Stirrup capacity in buckling was computed by secant formula and checked with field load tests. Percentage required for shrinkage and temperature changes is

made a function of the member's dimensions to check with established practice both in thin and massive members. In columns, tie spacing is governed by buckling of longitudinal reinforcement (which is a function of its yield point and of the angle subtended by the stirrup in each bent) and by shrinkage requirements. Paper includes tables (in metric units) directly applicable to design.

Practical computation of creep deflection of reinforced concrete beams (in German)

A. Habel, *Bautechnik* (Berlin), V. 34, No. 2, pp. 64-66

Applied Mechanics Reviews Nov. 1959 (Vasarhelyi)

Presents a conventional computational method for creep deflections of reinforced concrete beams. Graphs and tables provide for the selection of creep constants for various types of concretes and various reinforcement ratios (based on German specifications).

Theory of live load in buildings (in Spanish)

E. Rosenblueth, *Revista Ingenieria* (Mexico), V. 29, No. 4, Oct. 1959

Author's Summary

Proof is offered that, for sufficiently large loaded areas, the design unit live load is of the form $w = w_o (1 + cf/\sqrt{A})$, where w_o depends only on the intended use of the floor considered; c is a parameter depending on the shape of the influence surface associated with the generalized stress for which one is designing; f (in units of length) depends on the intended use of the floor and on the allowable probability of failure; and A is the loaded area. Several examples are considered, including statically determinate members, such as simply supported beams with overhangs (for which charts are provided), and elastic statically indeterminate members. The theory is generalized to limit design of beams and slabs. It is shown that $c = 1.2$ covers most practical applications. Data of office buildings cited by J. F. Baker (*The Steel Skeleton*) substantiate the proposed formula for loaded areas as small as 10 sq ft in office buildings. Statistical study undertaken by the Institute of Engineering, University of Mexico, to obtain quantitative information on parameters w_o and f, will be included in future paper.

Reflections on grids with edge and interior girders having different stiffnesses (Betrachtungen uber Tragerroste mit Steifigkeitsunterschieden zwischen Rand- und Innentragern)

Konrad Sattler, *Der Bauingenieur* (Berlin), V. 34, No. 1, Jan. 1959, pp. 1-9; No. 2, Feb. 1959, pp. 53-59

Reviewed by Aron L. Mirsky

Taking as a point of departure the results obtained by Guyon and Massonnet for the case of edge and interior girders of identical stiffness ($I_r = I_m$) [*cf*, author's paper in title source, V. 30, No. 3, Mar. 1955, pp. 77-89; "Current Reviews," ACI Journal, V. 28, No. 6, Dec. 1956 (*Proceedings* V. 53), pp. 612-613], and using a combination of symmetric and antisymmetric components to obtain the case $I_r \neq I_m$, author develops an approximate method for solution of grids with various numbers of girders and with and without torsional resistance. Major portion of paper is devoted to numerical examples, 12 in all.

Hinges in structures (Les rotules dans les structures)

Michel Normandin, *L'Ingenieur* (Montreal), V. 45, No. 178, Summer (June) 1959, pp. 29-32, 36

Reviewed by Aron L. Mirsky

The term "hinges" here refers to points of zero moment in hyperstatic structures. For the relatively simple case of uniformly distributed loads covering entire spans, author develops formulas relating the positions of the points of zero moments, the ratio of dead and live loads, the ratio of maximum and minimum moments, and the span lengths. These formulas can then be used to determine span lengths

which will yield, say, equal positive and negative moments, when given the total length of the structure and the number of spans desired. Two major cases are investigated: two unequal spans, and several unequal spans, with the latter case being treated more fully for structures of three and four spans. Numerical examples are given.

Contribution to the analysis of elastic and plastic deflections of reinforced concrete beams with mild-steel reinforcement (Ein Beitrag zur Berechnung der elastischen und plastischen Durchbiegungen schlaff bewehrter Stahlbetonbalken)

A. Mehmel, *Der Bauingenieur* (Berlin), V. 34, No. 1, Jan. 1959, pp. 9-12

Reviewed by Aron L. Mirsky

Develops expressions for and curves of correction factors, with effects of shrinkage and creep included (but with E considered to be unaffected by time), to be applied to evaluations of beam deflection per German standard DIN 1045. Author notes that a limiting value for total deflection, for inclusion in specifications, is still to be developed. Numerical example is included.

Buckling analysis of reinforced concrete frame structures

L. Palotas, *Scientific Publication of the Technical University* (Budapest), 1957, pp. 13-27

Applied Mechanics Reviews Nov. 1959 (Goldberg)

Presents three approximate methods for determining critical loads of rigid-jointed frames with particular reference to frames of reinforced concrete. In the first method rotational end restraints provided for critical members by adjoining members are estimated in terms of flexural stiffnesses and carry-over factors of these members. Effective length of critical member is then calculated approximately through use of the end-restraint factors. The second method considers entire frame and requires computation of transverse displacement resulting from unit lateral load. Unit lateral load is applied at joint if sidesway is assumed to be involved in buckling mode, or at or near point of maximum deflection of critical member if sidesway is not involved. The deflection due to unit load is also computed assuming that joints are locked against rotation. Since effective length is known in the latter case, comparison of lateral displacements in two cases provides means of estimating effective length in actual case.

In the third method, approximate buckling mode of frame is obtained by starting with a system of hinged bars, then restoring continuity. Critical loading corresponding to approximate mode is calculated by application of energy method.

Design of slabs for rectangular tanks

G. P. Manning, *Concrete and Constructional Engineering* (London), V. 54, No. 8, Aug. 1959, pp. 279-282

Presents charts to aid in the design of walls and bottoms of rectangular containers of liquids and which can be used when the bending moment causes tension in the face in contact with the liquid. The requirements and stresses are in accordance with proposals for a British Standard Code for such structures.

Analytical investigation on the effect of the rigidity of the boundary on the stresses all over the crossing beams of panelled floors

A. K. Labib, *Bulletin*, Faculty of Engineering, Cairo University (Giza), 1956-1957, pp. 69-80

Every beam in a paneled floor is liable to different stresses according to its position in the layout. The rigidity of the boundary has an enormous effect on the distribution of stresses over the different beams. This paper aims to relate the rigidity of the boundary with that of the crossing beams such that the beams will be liable to the same stresses regardless of their position in the layout. This enables using

the same cross section for all beams to satisfy the architectural point of view while satisfying the economical point of view.

Three different methods are applied to find the stress distribution over the structure; all are based on the elastic theory. By the first, the indeterminate structure is solved by satisfying deflection conditions. By the second, the problem is solved by influence coefficients. By the third, the similarity of paneled floors with flat slabs is taken into consideration and the usual empirical formulas of flat slabs are applied to the problem. Practically the same results are obtained by the three methods.

The first method is then applied, varying the rigidity of the bounding girders, watching its effect on the stress distribution, and finding out the appropriate rigidity at which all the crossing beams are subject to the same stresses.

Design of corner columns

R. J. Bartlett, *Concrete and Constructional Engineering* (London), V. 54, No. 9, Sept. 1959, pp. 293-301

Applies load-factor method to corner columns subjected to axial load and bending in two directions, usually at right angles. The solution gives different amounts of steel at each corner of a rectangular section. However, member will not be weakened if there is additional steel at one or more corners to achieve symmetry or to allow for more than one condition of loads. Results are not claimed to be exact but are judged to be reasonable.

Ultimate strength of long reinforced concrete columns eccentrically loaded

W. S. Hanna, M. Bakhoum, and L. Riad, *Bulletin*, Faculty of Engineering, Cairo University (Giza), 1956-1957, pp. 33-67

An extension to a previous paper (*Bulletin*, 1954) by the authors entitled "Axial and Eccentric Buckling with Special Application to Reinforced Concrete." That paper developed methods for the determination of stresses under a given load; the methods were verified experimentally.

In the present investigation three methods were developed for the determination of the ultimate capacity of eccentrically loaded long columns. The methods are applicable for homogenous elastic materials as well as for reinforced concrete.

The first method depends on a theoretical analysis that uses as accurate assumptions as is practicable. The second method is based on an approximate formula which gives the deflection of the eccentrically loaded column as a function of the original deflection and the ultimate strength of the member when axially loaded. The third method is a practical approach which is based on simplified approximate assumptions.

The investigation is preceded by a limited study for the case of axially loaded long columns which is used as a basis for the second method, and a proposed solution is presented.

Design of helical staircases—Statically indeterminate cases

Jacques S. Cohen, *Concrete and Constructional Engineering* (London), V. 54, No. 6-9: June 1959, pp. 197-204; July 1959, pp. 249-256; Aug. 1959, pp. 283-288; Sept. 1959, pp. 313-321

States general equations for equilibrium, deformation, and angular rotation for use in design of statically indeterminate helical staircases. Examples show how the complex equations are applied to design problems.

On analysis of built-up slabs

K. K. Banerjee and M. M. Basole, *Journal of Technology*, Bengal Engineering College (Calcutta), V. 3, No. 2, Dec. 1958, pp. 119-136

Built-up slabs are considered to be jointed, precast concrete units forming a slab with simple tongue-and-groove or mortar keys for joining the units. The authors have used Fourier's series to solve the problem in which joints between units carry no transverse moment but the distribution of

load is affected by transference of shear across the joints. The deflections of any two units at their common joint are the same and a set of simultaneous equations are developed and solved to provide the answers for deflection, moment, and shear.

Experimental values for moment and deflection obtained in this paper compare well with the calculated values or the evaluation of which two or three terms only of the series have been used.

Proceedings of the second symposium on concrete shell roof construction, 1957

Teknisk Ukeblad, Oslo; Claude Gill Books Ltd., London; 1958, 382 pp., £5

THE STRUCTURAL ENGINEER
Apr. 1959

The proceedings are divided into four sections: (1) recent notable shell structure designs—nine papers describing structures developed in Europe and America; (2) new trends in shell design, containing 16 papers on different types of shells and special design problems; (3) shell research, containing five papers on testing, etc.; and (4) two papers on prestressed and precast shells. Discussions and an evaluation of the results of the symposium are also included in the volume.

Influence lines for continuous beams

KERRY S. HAVNER and JAN J. TUMA, *Publication* No. 106, Oklahoma Engineering Experiment Station, Sept. 1959, 50 pp. $1.50

AUTHORS' SUMMARY

The application of the algebraic carry-over moment method to the analysis of multispan continuous beams of constant or variable cross section is presented. The general three-moment equation is the mathematical model used and the infinite, convergent, geometric series the means of solution.

The derivation of the three-moment equation for beams of variable section, the theory of algebraic carry-over procedure, and the derivation of moment influence coefficients in algebraic form are shown in Chapters 1 and 2. Numerical examples illustrating the application of these influence coefficients are included in Chapter 3. The appendix contains two sets of moment influence tables which are especially suitable for the analysis of continuous beam bridges.

Problem of the carrying capacity of light reinforced-concrete structures when crack formation takes place (in Russian)

V. K. BALAVADZE, *Soobshcheniya Akademii Nank Gruzin SSR* (Tiflis, USSR), V. 17, No. 8, 1956, pp. 737-744

APPLIED MECHANICS REVIEWS
June 1959

Reports on the use of an empirical formulas for determining the limits of crack width in ferro-concrete structures. Formula involves the relation between rod diameter and distance between rods.

Statically indeterminate structures—approximate analysis of deflected structures and lateral load analysis

J. R. BENJAMIN, Civil Engineering Series, McGraw-Hill Book Co., Inc., New York, 350 pp., $11

Reviewed by J. J. POLIVKA

This book is of great value both as a textbook and guide in engineering practice. It is largely concerned with the approximate analysis of statically indeterminate structures. In Chapter 1 the author emphasizes the fact that all structural analyses and designs are approximate, due to many assumptions which in reality do not comply with actual conditions. Often assumed dimensions, which are so important for statically indeterminate structures, must be changed in accordance with the results of the first analytical procedure, so the initial approximate analysis saves considerable time.

Chapter 2 reviews structural mechanics, i.e., statical indeterminacy, deformations, preliminary deformation models, and elastic and plastic behavior. Chapters 3, 4, 5, and 8 thoroughly discuss analytical procedures under vertical loads, and the effects of lateral

loads are covered in Chapters 6 and 7, also in which rigid frame structures and shear-wall structures are studied. Chapter 8 presents approximate analysis of miscellaneous structures like bents, bents with sloping and inclined members, gabled bents, and arches. Chapters 9 and 10 deal with some aspects of the philosophy of structural engineering, emphasizing the relationships between analysis and design. Of special interest to students and engineers are methods of checking the analyses, which many books overlook or only superficially mention. Chapters are amply illustrated, explained by numerical examples, contain problems to be solved by the readers, and includes important references. Reviewer believes that attention should be given to some time-saving analytical methods.

Computation of continuous beams with constant moment of inertia in each span with the help of Anger's tables (in German)

G. Woller, *Bautechnik*, V. 35, No. 5, May 1958, pp. 182-186

Reviewed by J. J. Polivka

Anger's tables, in V. 3 of his book "Zehnteilige Einflusslinien", are calculated for various ratios of moments of inertia $i = I_c/I_n$ and of spans $m = L_n/L_c$ in which ratios the n refers to individual spans and c to one characteristic span (either to equal end spans or to any span of the continuous beam assumed to be constant). The product of these coefficients is designated $k = mi$. Tables are calculated for so-called "elastic spans" $L_n - kL_c$, assuming $L_n = L_c$ (actual span). Author discusses these relations and proves that Anger's tables are correct for approximately equal coefficients k of all spans and for concentrated loads. Resulting moments at the supports are to be multiplied by I_n/I_c. Main object of the article is to introduce a method for using Anger's tables with sufficient accuracy for any type of loading or influence lines.

Prediction of temperature rise in fire testing of walls and partitions

R. H. Neisel, *ASTM Bulletin*, No. 239, July, 1959, pp. 63-65

Empirical formulas have been developed whereby the fire rating of wall panels may be predicted from the thickness, density, and thermal conductivity of the wall material or materials. The prediction relates only to the temperature-rise rating and not to structural integrity, which is primarily a function of the method of assembly. Method of prediction is applicable to a wide range of materials.

Materials

On the fineness and even size distribution of a ground material (Uber die Feinheit und Gleichmassigkeit eines Mahlgutes)

A. Kneschke, *VDI Zeitschrift* (Dusseldorf), V. 101, No. 11, Apr. 11, 1959, pp. 413-418

Reviewed by Aron L. Mirsky

Mathematical study, utilizing the residual function. Formulas derived are stated to apply to any material which can be ground and are particularly applicable to Rosin-Rammler materials.

Petrography of cement and concrete

L. S. Brown, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 1, No. 3, Sept. 1959, pp. 23-34

Author's Summary

Conventional petrography has developed as a technique for the study of rocks, mostly by viewing thin sections under a light-polarizing microscope. The technique has served well in elucidation of mineral composition, structure, origin, and history of natural rocks. In application to study of portland cement and clinker, certain unique features of the material have made necessary extensive supplement by the metallographic microscope and study of polished sections. In combination with this advantage, the technique of thin sections has been superseded largely by the easier preparation of powder mounts. Application to prob-

lems of concrete has required further broadening of the technique. The stereoscopic binocular is found to be the generally more useful and informative microscope, with aid of the petrographic and metallographic microscopes where higher magnification is specifically required. The amplified technique has proven of great advantage in working out problems in cement and concrete.

Rheology of disperse systems

C. C. Mill (Editor), Pergamon Press, Inc., New York, 1959, 224 pp., $9.50

Contains the proceedings of a conference sponsored by the British Society of Rheology. The papers fall into two groups. In one, the dispersions studied were dilute or composed of noninteracting particles and experiments were designed to test hydrodynamic theory. In the other group, concentrated suspensions as used industrially, mainly in various forms of surface coating, were studied with the object of relating performance to rheological properties. This book will be of some interest to cement chemists.

Effect of hydraulic additives on the kinetics of sulfate accumulations and corrosion processes in cements

YU. T. Tashpulatov and F. L. Glekel, *Zhurnal Prikladnoi Khimii*, V. 28, No. 8, 1955, pp. 822-830

Ceramic Abstracts Sept. 1959 (Palmeri)

Recent investigation of cements and clays shows that, contrary to former belief, hydraulic admixtures may be used to obtain sulfate-resistant cements. Formerly, one of the chief objections to the application of these mixtures was the formation of additional quantities of sulfoaluminates during cement hardening processes by the reaction of "active" alumina clays. The authors studied the kinetics of sulfate accumulations in portland cements containing admixtures, and the effect of these factors was observed when specimens were treated in solutions of different SO_4 concentration. The composition of selected specimens was $SiO_2$21.2,$Al_2O_3$5.86, $Fe_2O_3$3.59, CaO 66.40, MgO 1.62, and $So_3$1.33 percent; 20 to 40 percent mixtures of clay were added (obtained from kaolin and tripoli calcined at 800 deg), the composition of which was $SiO_2$60.19, $Al_2O_3$26.48, $Fe_2O_3$0.95, $TiO_2$0.52, CaO 1.80, MgO 0.94, and SO_3 0.14 percent. The specimens were hardened in water and 5 to 15 percent solutions of Na_2SO_4, and corrosion processes were studied during storage periods of 14 days to 1 year. Cements containing 20 and 40 percent admixtures of kaolin formed approximately 1.5 to 2 times more sulfoaluminate crystals than cements of the same composition containing 20 and 40 percent tripoli. There were definite indications of reactions of "active" alumina clays and the formation of additional quantities of sulfoaluminates. Final GOST-49 tests on hydro-industrial concrete prove that the addition of 50 percent mixtures of clays (kaolin or tripoli) to portland cement containing 9 percent tricalcium aluminate gives improved cement that has high resistance to sulfates and is free from concentrations of H_2SO_4. The failure of portland cement without these admixtures was caused by the formation of sulfoaluminates and crystals of $CaSO_4 \cdot 2H_2O$.

Relationship between strength and granulometric composition of cements, as based on statistical investigations (Zusammenenhang zwischen Festigkeit und Kornaufbau von Zementen auf Grund statischer Unterschungen)

H. Gebelein, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 9, Sept. 1959, pp. 423-429

Author's Summary

In air separation tests with two different initial cements, 48 different classification products with characteristic differences in grading were obtained. Statistical investigations of the results for these 48 cements yielded an empirical formula for the relationship between fine particles (< 10 u), medium sized particles (10-40 u), and coarse particles (> 40 u) and the

Blaine value. Strength tests with some of these cements furthermore showed how the strengths are related to the various particle size fractions, to the Blaine values, and also to the gypsum content.

A new method for accelerated testing of portland cements in respect to their strength development (Eine neue Methode zur beschleunigten Prufung der Portlandzemente auf ihre Festigkeitsentwicklung)

E. Brandenberger, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 9, Sept. 1959, pp. 385-392
Reviewed by H. H. Werner

Conditions for accelerating hydrothermal hardening of standard mortar prisms in an autoclave were ascertained, yielding comparative compressive strengths within ±10 percent, and almost certainly within ±20 percent, of the standard 28-day strengths. This provides a simple strength prognosis within 36 hr and the method has been in current use in the Swiss Federal Laboratories.

In joint investigation with seven other laboratories it proved to be readily reproduceable and fully satisfactory in more than 3000 cases. Exhaustive tests on experimental, "abnormal" cements were carried out to ascertain the validity of the new method.

The effect of additives on the corrosion behavior of steel in concrete (Die Auswirkung von Betonzusatzmitteln auf das Korrosionsverhalten von Stahl in Beton)

A. Baumel, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 7, July 1959, pp. 294-305
Reviewed by H. H. Werner

From current-voltage curves derived from electrodes embedded in concrete it can be inferred whether steel in concrete is liable to corrode. Twelve commercial additives investigated showed that only fairly large quantities of chloride additives have adverse effects. Current density-time curves with constant voltage are also suitable for investigation of corrosion characteristics.

Satisfactory and lasting corrosion protection is insured only by a complete cement-gel coating of the steel which provides a constant alkaline film of liquid on the steel surface, thus keeping the steel passive. An adequate cover of dense concrete is required, in turn, to protect the cement gel at the steel from external chemical or mechanical destruction.

The current-voltage curves can establish whether the steel in concrete is entirely passive or not and thereby provide a means of deciding whether stress corrosion is liable to occur.

Testing corrosion danger of steel reinforcement due to admixtures in concrete (Die Prufung der Korrosionsgefahrdung von Stahlarmierungen durch Betonzusatzmittel)

H. Kaesche, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 7, July 1959, pp. 289-294
Author's Summary

Under normal circumstances steel reinforcement in wet concrete is well protected from corrosion by a thin film of oxide (passive layer). There are, however, chemical compounds, especially calcium chloride, which destroy this protective layer and which may give rise to serious corrosion of the reinforcement. In the case of prestressed concrete structures, in particular, care must be taken to ensure that no aggressive compounds are introduced by the use of unsuitable admixtures, inasmuch as the corrosion of prestressing wires may assume the dangerous form of stress corrosion.

The existence of a stable passive layer around reinforcing bars can be demonstrated with the aid of current-voltage curves for steel bars embedded in concrete cylinders and exposed to the action of lime milk. The method is suitable for testing the aggressiveness of admixtures of unknown composition. A testing procedure which has been developed for carrying out routine measurements of this kind is described.

Curing of mortar with infrared rays (in Japanese)

AKIRA SAKAMURA and GOSEKI YAMAMOTO, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 124-28
CERAMIC ABSTRACTS
Sept. 1959 (Suzukawa)

The effect of initial curing at 60, 80, and 100 C, for 1 to 3 hr in a room with an infrared lamp on the strength development of mortar was tested. The specimens cured at 60 C, at the age of less than 2 hr after molding showed a high strength. It is desirable to prevent the loss of H_2O during irradiation with infrared rays by covering the mold with a glass plate.

Study of the utilization of fly ash in building

P. BRZAKOVIC, *Nase Gradevinarsto* (Belgrade), V. 13; Part 1, No. 2, Feb. 1959; Part 2, No. 3, Mar. 1959, pp. 42-47; Part 3, No. 4, Apr. 1959, pp. 86-92
Reviewed by J. J. POLIVKA

Presents a thorough description of all characteristic properties of fly ash in cement and lime mixtures, especially those used for the production of lightweight concrete. A relatively unknown fact mentioned is that the Yugoslav lignites have similar chemical properties as other European anthracite coals. Pozzolanic effects and the influence of different factors, such as fineness, carbon content, and water-cement ratio are discussed in Part 2. Characteristics of concrete made with fly ash, such as its influence on strength, resistance to weather conditions, especially to frost and corrosion, are discussed. The study includes shrinkage, heat of hydration, alkali-reaction, and waterproofing of these concretes. Part 3 refers primarily to lightweight concrete manufactured in Yugoslavia, especially those utilizing fly ash from the power station in Kostolac, Serbia. It is emphasized that this fly ash, obtained by combustion of lignite, has similar properties to European anthracite coals. Investigation of lightweight concrete using this type of fly ash shows that good quality can be obtained from a mixture of 70 percent fly ash and 30 percent of cement lime. Such concrete weighs 71 to 82 lb per sq ft after treatment in the vacuum press. Insulation properties are about the same as lightweight concretes with light aggregates.

Styropor concrete (Styropor-Beton)

H. EICK, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 6, June 1959, pp. 253-257
Reviewed by H. H. WERNER

Foamed polystyrene was investigated with regard to its suitability as a lightweight aggregate. It was found that foaming of polystyrene can most conveniently be effected in boiling water which results in a dry bulk density of 17 kg per cu m (1.06 lb per cu ft).

Test series using neat cement and cement and sand were made to establish mixing qualities, densities, strengths, absorption, and conductivity. Two suitable mixes are formulated and full-scale tests made, establishing the possibility to use Styropor concrete for insulating blocks and planks.

Water-retardant qualities of external coatings

I. SOROKA, *ASTM Bulletin*, No. 240, Sept. 1959, pp. 40-45

Tests were carried out to develop a procedure for the comparative rating of water-retardant qualities of external coatings under laboratory conditions. Thirteen coatings of different makes were tested using a modified chimney method under constant pressure, equivalent to 100 mm head of water. Coatings were applied to sand-lime bricks and the amount of water penetration was recorded daily for periods up to 23 days. Procedure was repeated twice for each specimen, before and after application of the coating. Comparative rating was based on performance's dependence on the properties of the untreated specimens, as well as on its retardant qualities indicated by keeping penetration below a certain level .

The article discusses three general groups of existing test methods—artificial rain method, flat immersion method, and the chimney method—then describes this proposed test procedure

with results and evaluations. This proposed procedure apparently gives a reliable means of comparative rating of the water-retardant qualities of external coatings, based on the actual reduction of water penetration. The results indicate the higher quality of silicone coatings in gasoline solutions. Sodium silicate solution was the poorest water-retardant coating tested.

Heavy-media separation passes the test

Contractors and Engineers, V. 56, No. 6, June 1959, pp. 60-63

Describes the general process and procedure followed in providing the Glen Canyon Dam with the required 800 tons per hr of clean concrete aggregates. Tells in particular why heavy-media separation was used.

Foam concrete in construction (in Rumanian)

F. Calinescu and N. Stanescu, *Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 12, Dec. 1958, pp. 607-613
Reviewed by J. J. Polivka

Thoroughly reviews experiments, studies, and applications of foam concrete precast structural units cured by steam or treated in autoclaves at 8 atmospheres pressure. Also describes structures cast on the site and cured in the usual way.

Studies of calcium sulfoaluminate admixtures for expansive cements

A. Klein and G. E. Troxell, *Proceedings*, ASTM, V. 58, 1958, pp. 986-1008
Authors' Summary

Various clinkers of theoretical calcium sulfoaluminate composition were produced and then studied by chemical analyses, differential thermal and x-ray analyses, and petrographic examinations to determine their compound composition. It was established that the raw materials did form a new compound not precisely identified, but which most likely was an anhydrous calcium sulfoaluminate.

A clinker burned at 2480 F, when ground and blended with slag and portland cement and used in 1:2.75 graded Ottawa sand mortars, cured 6 days in water and then dried in air 120 days developed final net expansions which ranged from 450 to 1800 millionths per unit. For a corresponding clinker burned at 2550 F, the final net expansions ranged from 5000 to 10,000 millionths per unit. In each case the coarser admixture showed the greatest net expansion. Somewhat smaller expansions in concrete may be sufficient for the "self stressing" of prestressed concrete.

The compressive strengths developed by some of the sulfoaluminate expansive cements were satisfactory for structural concrete.

Setting and hardening of hydraulic materials. I—Setting and hardening of Paris plaster

I. Naray-Szabo and G. Szuk, *Acta Technica*, Academiae Scientiarum Hungaricae (Budapest), V. 22, No. 1-2, 1958, pp. 67-82

Setting and hardening of hydraulic materials. II—Conductometric analysis of setting of cement paste under isothermic conditions

G. Szuk and I. Naray-Szabo, *Acta Technica*, Academiae Scientiarum Hungaricae (Budapest), V. 22, No. 1-2, 1958, pp. 83-100

Reports on studies of the setting and hardening of portland cement and other hydraulic materials. The first report is concerned with an investigation of gypsum pastes of different water content.

Investigations into the setting of different cements, with a conductometric method, reported in Part II, showed the existence of three pronounced minima in the resistance-time curves of cement pastes. The temperature of cement paste in cubic Vinidur cells of 2 cm edge and immersed in an ultrathermostat of 20,000 ± 0.001 C has been measured against time. The second resistance minimum seems to indicate the beginning and the third the end of setting, both of them being more readily observable and sharper than by the classical Vicat test, with

which a comparison has been also made. The changes in temperature, pH value, and resistance have been measured and are discussed from the physico-chemical point of view.

Epoxy resins for use on civil works projects. Summary of data available as of March 1, 1959

LEONARD PEPPER, *Technical Report* No. 6-521, Report No. 1, U. S. Army Engineer Waterways Experiment Station, Aug. 1959, 47 pp. $0.75

AUTHOR'S SUMMARY

Reviews the available literature and manufacturers' publications to determine the potential uses of epoxy resins on civil works projects. These possible uses fall into three general categories: adhesives, fillers, and coatings. Specific applications that have proved successful include: (a) bonding hardened concrete to hardened concrete; (b) bonding freshly mixed concrete to hardened concrete; (c) sealing cracks in concrete; (d) filling voids, pop-outs, and spalls in concrete; (e) refacing worn concrete or steel surfaces; (f) coating concrete or other surfaces for resistance to chemical attack; (g) protecting electrical components; and (h) bonding attached or embedded items such as dowels, reflector buttons, and traffic bars to concrete. It is recommended that investigation of epoxy-resin systems be continued to: (1) define the uses that are most economically attractive; (2) develop specific performance specifications applicable to such uses; and (3) develop application procedures and information on performance in service.

Franciscan chert in California concrete aggregates

HAROLD B. GOLDMAN, *Special Report* No. 55, California Division of Mines, 1959, 28 pp., $0.50

Treatise on the nature and determination of the chemical and physical properties of Franciscan chert that affect the use of this material in portland cement concrete. Data from laboratory tests which demonstrate that the Franciscan chert is moderately reactive are presented and evaluated. Results of field examinations of concrete structures built with sand and gravel, containing varying proportions of Franciscan chert, revealed no signs of alkali-aggregate reaction after 20 to 50 years. A table contains data on the distribution of the chert in the major stream deposits of the Coast Range.

Aeration cause of false sets in portland cement

W. C. HANSEN, *Proceedings*, ASTM, V. 58, 1958, pp. 1044-1058 (including discussion)

AUTHOR'S SUMMARY

Evidence, based on extractions with water and the rates at which cements combine with calcium chloride, is presented to show that cements which develop false set on aeration do so because the reactivity of the $3CaO \cdot Al_2O_3$ is decreased sufficiently to permit dehydrated gypsum in the cement to precipitate as gypsum in the cement-water paste.

Reflectance tests for concrete curing materials

C. C. RHODES, M. H. JANSON, and M. G. BROWN, *Proceedings*, ASTM, V. 58, 1958, pp. 1059-1076 (including discussion)

AUTHORS' SUMMARY

Stress control through the regulation of early temperature changes in concrete is an important function of the curing operation. Since the primary purpose of using white curing materials is to reduce heat pickup from solar radiation, the measurement of reflectance is of considerable significance in procuring such materials. Because existing preference is divided between the luminous reflectance method of ASTM E 97 and the infrared reflectance method of Federal Specification TT-P-141b, this study was undertaken to provide a sound basis for evaluation of these two methods and development of reflectance tests in general.

Three factors enter into the development of a significant and practical

reflectance test for concrete curing materials: (1) spectral distribution of solar radiation received at the earth's surface, (2) suitability of various photometers for measurement of reflectance in the most significant spectral range, and (3) spectral characteristics of the curing materials. An analysis of these three factors indicates that measurements in the visible range are as significant as those in the infrared and that ASTM E-97 for routine testing and ASTM D 307 and D 791 for reference are the best that can be found at present using commercial photometric equipment currently available at a reasonable cost.

Properties of active volcanic tuffs and the possibility of their use in construction works

VLASTIMIR TUFEGDZIC, *Nase Gradevinarstvo* (Belgrade), V. 13, No. 2, Feb. 1959, pp. 30-35
Reviewed by J. J. POLIVKA

Thoroughly describes the hydraulic and pozzolanic properties of volcanic tuffs that are found in the Vranje valley. These tuffs were mixed with hydrated lime and then tested in the Belgrade University laboratory and subsequently used in construction and with great success and economy. Tuff-cement mixtures have good strength and their use for lightweight, precast, concrete units has growing application. Test results are tabulated and demonstrated in diagrams.

The setting of portland cement. A review of theory, performance, and control

H. H. STEINOUR, *Bulletin* 96, Portland Cement Association, Chicago, 1958, 124 pp.
ROAD ABSTRACTS June 1959

The literature on the subject is reviewed with special attention to factors affecting the setting times and development of false set. Chapters are included on the setting of ground clinker, the chemical action of gypsum, other retarders of clinker, the setting of gypsum-retarded cement, effects of calcium sulfate in other forms, the liquid phase in cement paste, altering the setting time of normally retarded cement, troubles with gypsum - retarded cement, remedial measures, and testing for early stiffening and early reactions in cement paste. The bibliography contains 231 references.

Capillary water absorption of blast-furnace cement mortar (Kapillare Wasserabsorption von Huttenzementmortel)

G. TSUNEYAMA, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 6, June 1959, pp. 260-264
AUTHOR'S SUMMARY

The capillary water absorption of blast-furnace cements, as compared with portland cement, has been measured by a method devised by the author. Blast-furnace cements containing 60-80 percent slag ("Hochofen" cements) displayed low absorption values, with more finely ground blast-furnace slag having even less absorption. The difference disappears with advancing age.

Aggregates for runway pavements

J. E. GRAY, *Crushed Stone Journal*, V. 34, No. 3, Sept. 1959, pp. 3-8, 23

Reviews a typical aggregate specification for portland cement concrete paving. States the performance expected of concrete runways and then discusses the probable modification of the usual requirements to assure satisfactory performance. Discusses ASTM C 33-57, "Standard Specifications for Concrete Aggregates," for requirements for grading, deleterious materials, soundness, and abrasion. Other properties which must be evaluated before a sound selection can be made are (a) the effect of coarse aggregate on the flexural strength of concrete, (b) the effect of coarse aggregate on the volume change of concrete, and (c) the effect of aggregate on skid resistance. In many cases, available aggregates must be used and although it is impractical to require the aggregates

to be free of all deleterious materials, the tolerance should be more restrictive for concrete runways than the current tolerance in highway construction. Suggests the possibility of two-course paving; e.g., the inferior aggregate used for the concrete in the lower part and then topped with concrete made with better aggregate.

Effect of type of capping material on the compressive strength of concrete cylinders

GEORGE WERNER, *Proceedings*, ASTM, V. 58, 1958, pp. 1166-1186 (including discussion)

A study was made of different types of capping materials permitted for use on concrete specimens by current ASTM test methods. The capping materials used were a neat high-alumina cement, a mixture of portland cement and plaster of Paris, two sulfur cements, two gypsum plasters, plaster of Paris, and sulfur without filler or additive. The study was conducted on cylinders with rough and smooth ends and with concrete cylinders of relatively low and high strength.

Effect of vacuum mixing on concrete containing surface-active agents

J. W. SKEEN, 31st International Congress of Industrial Chemistry, Liege, 1958, 4 pp. (Reprinted by Building Research Station (England) as Reference QL-622)

By mixing concretes containing different surface-active agents at a low pressure it was intended to prevent, or at least reduce, the entrainment of air by the agent. It was figured that this would allow seeing whether the agent still gives an improvement in workability of the mix from some physico-chemical effect other than from the presence of entrained air.

The results obtained with a limited number of agents are given. Within the fairly high experimental errors of the setup and the methods of test, the workability of concrete mixes as measured by the compacting factor was approximately proportional to the air content with certain surface-active agents irrespective of the amount of agent added. With certain other types of surface-active agent the workability was greater than could be accounted for by air entrainment alone.

Slip-form paver masters "split-level" grade

Construction Equipment, Sept. 1959, pp. 31-33

Reports on a new paving machine that replaced up to three conventional spreading and finishing machines. This machine applies the slip-form technique for forming curbs thereby eliminating all formwork and labor needed for this purpose. On this particular project 4000 to 10,000 ft of steel forms were eliminated and the labor force was reduced by 25 to 50 percent.

More speed for a paving train

BILL ALLEN, *Contractors and Engineers*, V. 56, No. 6. June 1959, pp. 60-63

Reports on two time saving devices which were employed in the paving of a 15½-mile section of U. S. 40 near Topeka, Kan. The first technique was to use the lead concrete spreader to tow a cart which carried the wire mesh reinforcing mats. The second innovation was a three-disc wheel mounted at the rear of the first spreader. This was used for placing short transverse tie bars down the center of the pavement. Both wheel and mesh carts helped cut costs. An added novelty on this project was the lime stabilized subgrade

Concrete pavement design research: 1958

Bulletin No. 217, Highway Research Board, May 1959, 49 pp., $1

Presents two papers on rigid pavement design research. "Fifteen-Year Report on Experimental Concrete Pavement Project in Oregon," by G. S. Paxson, deals with a portion of the northeast Portland secondary highway paved in 1941, with facilities for meas-

uring performance of expansion and contraction joint at various spacings, with and without load transfer devices.

"Performance of Doweled Joints Under Repetitive Loading," by Leslie W. Teller and Harry D. Cashell, describes load-transfer tests of full-sized joints under shear and flexure, and presents results of a series of tests to determine the effects of several variable factors.

Highway pavement design in frost areas, a symposium: Part 1. Basic considerations

Bulletin No. 225, Highway Research Board, Sept. 1959, 131 pp., $2.60

The information concerns the basic considerations of the frost action problem, namely, water temperature, soil, and the freezing mechanism of soil-water systems. The five papers comprising the discussion on basic considerations are: "The Mechanism of Frost Heaving in Soils," by E. Penner; "The Factor of Moisture in Frost Action," by P. F. Low and C. W. Lovell; "Frost Penetration: Relationship to Air Temperature and Other Factors," by M. S. Kersten; "The Factor of Soil and Material Type in Frost Action," by K. A. Linell and C. W. Kaplar; and "Frost Action in Soils; A Symposium Analysis," by Carl B. Crawford.

Accelerated proof-tests of runway pavement, Columbus Air Force Base, Mississippi

CHARLES R. FOSTER and J. P. SALE, *Proceedings*, Highway Research Board, V. 38, 1959, pp. 183-218 (including discussion)

AUTHORS' SUMMARY

Accelerated traffic was applied to pavements to proof-test the Army Corps of Engineers' design for interiors of runways under simulated B-52 aircraft traffic. A total of 5000 coverages was applied with a cart equipped with four tires arranged in the same pattern as the tires on the rear gear of a B-52. The tires were inflated to 266 psi and the cart was loaded to 212,000 lb. The paper describes the pavements, the accelerated traffic procedures, and the behavior of the pavements under traffic.

17-year report on the Owensboro-Hartford cooperative investigation of joint spacing in concrete pavements

MILTON EVANS, JR., and W. B. DRAKE, *Proceedings*, Highway Research Board, V. 38, 1959, pp. 226-240

This report provides data and information on subgrade, traffic, riding quality, and over-all condition from a 17-year performance survey on concrete pavement constructed by the Kentucky Department of Highways.

The following conclusions are made: (1) Expansion joints more than 400 ft apart are of little benefit and are probably detrimental. (2) Contraction joints, for best performance, should be closely spaced. (3) Dowel bars for load transfer at contraction joints are of questionable value except in the case of joints that open considerably and remain open. (4) A thickened edge pavement is actually superior to that of uniform 7 in. thickness.

Refractoriness of concrete airport runways (in Russian)

G. G. MUDROV and A. V. MIKHAILOV, *Beton i Zhelezobeton* (Moscow), No. 7, July 1959, pp. 318-320

Reviewed by D. WATSTEIN

Temperature measurements were made at various depths of concrete slabs and subgrades subjected to the combustion gases of jet aircraft engines. Surface temperatures of the concrete slab ranged from 200 to 400 C, and the durations of exposure ranged from 5 to 30 min.

The temperature distribution was determined in slabs 40 cm thick. The temperatures recorded 11 cm below the surface were not appreciably affected by the application of heat of 30 min duration. The temperatures at a distance 2 cm below the surface were found to be within limits which can be tolerated by ordinary portland cement concretes.

The refractive properties of 15 x 15 x 55 cm prisms were also determined by heating to 600 C in an electric furnace. The modulus of rupture

of limestone concrete was not appreciably affected by being heated to 600 C 15 times, while concrete containing granite aggregate lost half of its strength after five heating cycles, and all of its strength after 15 cycles.

Warping stresses and deflections in concrete pavements

M. E. Harr and G. A. Leonards, *Proceedings*, Highway Research Board, V. 38, 1959, pp. 286-320

A new theory has been developed whereby the stresses and deflections in concrete pavements subject to temperature and moisture gradients can be computed for the general condition that warping is sufficient to raise a portion of the slab off its support. The extent of the support can also be determined. Nomographs were prepared giving ratios of the critical stress to the modulus of rupture for a wide range of environmental gradients and for various combinations of slab sizes and thicknesses. Comparisons with available measurements showed good agreement between observed and computed deflections. Typical cracking patterns observed in concrete pavements can readily be explained.

The new theory is completely rational and has been extended to include a variety of superimposed loadings combined with the effects of warping produced by temperature and moisture gradients.

Final report on design project, Michigan test road

E. A. Finney and LeRoy T. Oehler, *Proceedings*, Highway Research Board, V. 38, 1959, pp. 241-285

This report completes the research phase of the design project on the Michigan Test Road. It contains miscellaneous project information pertaining to soil conditions, physical properties of concrete, climate, and traffic, which contribute to an understanding of the findings.

Approximately 45,000 joint width measurements were made on more than 850 joints during the 17-year study. With the information gathered, it was possible to compare various sections and determine the effects on contraction joint width of (a) expansion joint spacing; (b) intermediate warping joints; and (c) contraction joint spacing.

The physical performance of the pavement sections is discussed in terms of cracking, spalling, roughness, and joint faulting, to relate these to design factors such as joint spacing, pavement thickness and cross section, amount of reinforcement, and load transfer features at joints. Final observations are also given in one of the incidental studies—stress cured concrete pavement.

Concrete pavement: Subbase and joint construction

Bulletin 229, Highway Research Board, 1959, 53 pp., $1.00

Performance of concrete pavements as related to subbase construction methods

Phil Fordyce and J. D. Geesaman, pp. 1-10

Since 1952, the Portland Cement Association has been engaged in an extensive study of subbases for concrete pavements. Information was obtained during the construction of 28 projects in many parts of the country, representing typical subbase construction methods. The effects on subbase density and gradation of various construction operations are discussed. These include methods of subgrade compaction, subbase mixing, placement and compaction, and fine-grading.

The field studies of the performance of concrete pavements built on subbases of various designs and materials show that the method of subbase construction is an important factor. Uniformity of gradation was found to be the one subbase quality having the most influence on pavement performance. Subbase materials having a small maximum size were found generally to perform better than coarser materials because segregation is less likely

Note

Copies of books and articles reviewed are not available from ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In some cases ACI can furnish addresses of publications added later.

to occur during placement. Heavy construction traffic on the completed subbase nullifies the efforts expended to obtain uniformity and results in substandard pavement performance.

The report is concluded with a summary of nine construction procedures which have been found to contribute towards providing uniform pavement support.

Joint construction in concrete pavement
GORDON K. RAY, pp. 11-19

Research and design engineers have developed a vast storehouse of information on performance of various types of joints in portland cement concrete pavements. The performance of such joints depends to a large degree on the type and quality of their construction. Careless construction practices or incorrect methods of construction resulting from a lack of knowledge of the joint function results in pavements with below standard riding characteristics and maintenance problems.

The four general types of joints—longitudinal, contraction, expansion, and construction—are described, giving their purpose and design. Then, each joint type is taken up and satisfactory construction procedures outlined. In some cases, there are several different methods of construction which will produce a suitable joint. Wherever possible more than one acceptable method has been discussed. Included photographs demonstrate the various construction methods.

A theoretical approach to design of a road joint seal
EGONS TONS, pp. 20-53

A pavement joint undergoes continued changes in width due to such influences as temperature and moisture fluctuations within the concrete slabs. When a joint (or crack) is filled with a sealing compound, strains and stresses caused by every opening or closing movement must develop both within the sealer and along the bond interfaces between sealer and pavement. The mass of sealer in the joint can be pictured as a sort of bridge spanning the gap between two slabs. Since a bridge is designed before it is built by using known quantities and methods, an attempt should also be made to design a seal before it is placed in the joint.

The purpose of this study was to correlate mathematically and by experiment (a) the joint width, (b) sealed depth, (c) joint expansion, and (d) the extensibility of the sealer, and then hopefully to outline a joint seal design procedure for future practical use in the field. Work on this problem started with a simple joint model test. It was soon discovered that the depth of seal in the joint is another equally important variable, along with (1) amount of change in joint width, (2) capabilities of the sealing material, and (3) width of the joint space.

The paper describes the theoretical analysis and computations of strains in a sealed joint and presents experimental laboratory data to verify the basic theory and to point out its limitations. Nine sets of curves are included for illustration, which show that strains in the sealer can be decreased by either increasing the minimum joint width or by decreasing the depth of seal. The paper includes recommendations for additional research needed to broaden the scope of the basic theory and to further check the

limitations. Following this article are: Appendix A, Outline of a Bond-Ductility Test for Evaluation of Sealers; Appendix B, Probable Joint Seal Design Procedure; Appendix C, Discussion of Proposed Design Procedure; Appendix D, Test Road to Check Joint Seal Design Theory; Appendix E, Miscellaneous Equations.

Precast Concrete

Some recent developments in the technique of precasting concrete structures

J. A. Derrington, *The Reinforced Concrete Review* (London), V. 4, No. 1, Mar. 1956, pp. 1-31; discussion pp. 31-39
Reviewed by William R. Lorman

Describes and illustrates various structures recently erected in England. Presents erection problems encountered with precast beams, staircases, floor slabs, and shell roofs; also certain prestressing operations. Structures involve office buildings, printing works, factories, and electric power plant. Discussion by ten individuals pertains to American practices with floor slabs, Australian problems, efficacy of joint connections, and volume changes.

Development of grooved lightweight concrete blocks

T. L. Carhart-Harris, *Builder* (London), V. 196, No. 6063, June 12, 1959, pp. 1046-1047 (Reprinted by Building Research Station (England) as Reference QL-609)
Author's Summary

Advantages of grooved block and their design and development (making use of existing manufacturing sizes 18 x 9 x 4½ in. or 3 in.), the groove shape, and the adaptation of existing production are given. From only one manufacturing length of 18 in., any 3 in., or 4½ in. cut or their multiples can be made cleanly and accurately with two usable offcuts and thus walls can be built without waste to any 1½ in. multiple of length and can accept any intercepting header of 1½ in. multiples. Goes on to describe present site practice in cutting concrete block and how the grooves allow easy and accurate cutting of the block while eliminating three of the four usual operations required to cut a block accurately.

Faulty block curing practices

W. J. Shore, *Modern Concrete*, V. 23, No. 7, Nov. 1959, pp. 42-45

The author shows the wide variations in strength which occur due to improper atmospheric pressure steam curing practices for concrete block and describes simple tests to determine how curing methods of this type can be used to improve efficiency. After decribing the types of curing, he discusses research findings on steam curing practice and the effect on block strength of the method of steam curing. After demonstrating a need for improvement in curing techniques, he proposes a simple series of tests to check on the efficiency of cement utilization in a plant, on the relative uniformity of the curing processes, and on the reasons for unnecessarily high strength in the units of 24 hr age. With the data from these tests it is a simple matter to modify steam supplied to the curing kilns so that block on the top shelves of the racks are not subjected to excessive heat.

Use of precast and prestressed reinforced concrete constructions in industrial buildings in the USSR

B. F. Vasiljev, *Indian Construction News Supplement* (Calcutta), V. 8, No. 5, May 1959, pp. 11-26

The industrialization of buildings, consistently followed by the USSR, demands fabrication of building details and constructions at factories. Organization of factory fabrication of building elements is expedient only if there is a large repetition of elements and a restricted numbr of types and sizes. This standardization in building is carried out on the basis of the united modulus system, which sets rules for determination of geometrical parame-

ters of buildings, sizes of details, and rules of inter-coordination of details and elements of buildings with geometrical parameters.

Bays are regulated on the basis of 3-m modules. Load capacity of traveling cranes is regulated to specified weights; building heights are held to modules; three different roofing loads are established and five standard live floor loads.

On the basis of unified parameters of buildings and establishment of a unified system of distance coordination, the nomenclature and catalog of reinforced concrete buildings were worked out for industrial buildings with a limited number of types and sizes of various elements: columns, main carrying elements of coverings, secondary elements of coverings, foundation and framing beams, and crane beams.

In addition to discussing generally the ideas of standardizing the use of precast concrete by establishing particular modules, the article also details some of the particular precast members such as columns, trusses, and beams.

Refractory concrete moulds

E. HAMMOND, *The Engineer* (London), V. 207, No. 5393, June 5, 1959, pp. 878-882
Reviewed by ARON L. MIRSKY

Discusses refractory molds made with high-alumina and pure calcium aluminate cements, from standpoint of strength, porosity and permeability, thermal conductivity and diffusivity, expansion and shrinkage, thermal shock resistance, thermal stability, reproducibility, economics, and use in precision casting by various methods.

Automation in the concrete product factory

The Reinforced Concrete Review (London), V. 4, No. 1, Mar. 1956, pp. 59-68
Reviewed by WILLIAM R. LORMAN

Describes operations (illustrated in 14 photographs) and details of mechanical processes employed in USSR factories producing prestressed precast concrete units for use in prefabricated structures in Moscow. Each plant uses 160,000 cu yd of concrete annually and the four production lines in each plant provide units such as beams, columns, hollow floor slabs, and wall panels; but there is no indication of the number of such plants involved and therefore the reader has no idea of the total magnitude. The article ends with a description of how the various concrete units are used in the erection of apartment dwellings.

Prestressed Concrete

Research in prestressed concrete at the University of N. S. W.

A. S. HALL, *Constructional Review* (Sydney), V. 32, No. 5, May 1959, pp. 18-24

A method of estimating the shear strength of simply supported beams without web reinforcement is given. This applies to T and I beams as well as rectangular beams. Bond tests on both smooth and indented wires are reported. The effect of creep on the time behavior of redundant reactions in continuous beams is discussed. Associated with the last topic, measurements have been made of creep and of stress relaxation which are found to be related.

Prestressed concrete tank for the Canada-India reactor

Indian Concrete Journal (Bombay), V. 33, No. 8, Aug. 1959, pp. 262-264

The first prestressed concrete tank is now being built in India by the Atomic Energy Commission as an emergency water storage tank for the Canada-India reactor. This is a rather unusual looking structure. The reservoir itself is a sphere with outside diameter of 72 ft 5 in., resting on a conical reinforced concrete tower 20 ft above the ground level. Its capacity is 850,000 gal. The top of the reinforced

conical tower which supports the tank has an inside diameter of 40 ft 8 in. at this level. The tower rests on a reinforced concrete footing founded on medium hard rock 22 ft below the ground level. The center line of the footing describes a circle of diameter 53 ft 4 in.

The sphere comprising the tank has an outer radius of 36 ft 2½ in. The wall thickness is 7 in. in the upper and middle parts and increases progressively to 13 in. in the lower part near the lower ring beam. The wall is prestressed by horizontal and vertical Freyssinet cables made up of 12 high tensile steel wires, 5 mm in diameter with a minimum ultimate strength of 212,000 psi. The spacing of the 104 horizontal prestressing rings varies from the bottom to the top in accordance with the magnitude of water pressure. The number of vertical prestressing cables is 92. These are placed along meridian lines and are anchored at the bottom in the lower ring beam and at the top in the upper ring beam. Four pilasters which contain the anchorages of the horizontal prestressing cables are located in two vertical meridian planes at right angles. The dome of the roof is an extension of the sphere of the tank. It is 5 in. thick and has an inside radium of curvature of 35 ft 7½ in.

Effect of temperature drop on strand stresses in a casting bed

H. KENT PRESTON, *Journal*, Prestressed Concrete Institute, V. 4, No. 1, June 1959, pp. 54-57

Author discusses the problem of strand failures encountered in the fabrication of pretensioned prestressed concrete members when there is an appreciable temperature change during the curing period and before the strands have been unjacked.

A graph is introduced so as to help in determining what temperature drop will induce dangerous tensile forces.

Prestressed concrete poles for power transmission lines

R. P. MHATRE, *Indian Concrete Journal* (Bombay), V. 33, No. 8, Aug. 1959, pp. 265-269

Advantages are cited for prestressed concrete poles for transmission lines: Prestressed concrete poles are more durable than wood, steel, or reinforced concrete poles; they weigh considerably less than reinforced concrete poles; they are safer for all voltages as compared to steel poles; they are economical in the consumption of steel; the cost of maintenance is almost nil; they require no concrete foundation; and the cost can be pared down considerably by standardization and simplification of design. The manufacture, design, handling, transportation, and erection of these poles are discussed. Drawings show details of square, rectangular, I-shaped, and Vierendeel prestressed concrete poles, and a composite pole. The author concludes that the precast pretensioned poles can compete with wood, steel, or reinforced concrete poles when consideration is taken not only of the initial cost but the cost of maintenance and foundation blocks as well, and the greater durability of the prestressed poles.

Prestressed concrete as applied to buildings

Indian Construction News (Calcutta), V. 7, No. 8, Aug. 1958, pp. 17-136

The proceedings of a symposium "Prestressed Concrete as Applied to Buildings" held at Central Building Research Institute, Roorkee, India, Feb. 10-12, 1958. Twenty-one papers are presented under the major headings of: materials and methods, prestressed concrete roofs, research on prestressed concrete as applied to buildings, and economics of prestressed concrete as applied to buildings. Various individual papers will be reviewed in the "Current Reviews" section of the JOURNAL as space permits.

Use of prestressed concrete girders for power plant roofs

ERIK SOLLID, *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 65-67 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

AUTHOR'S SUMMARY

A comparison between the conventional steel roof trusses and prestressed concrete girders carried out for a power plant roof, for which the frame analysis and steel truss design were available, brings out clearly the saving in steel and construction cost resulting from the use of prestressed concrete.

The span is 61 ft and the roof is 372 ft long requiring 23 girders, the maximum center-to-center distance between girders being 18.5 ft.

Prestressed concrete as applied to buildings

N. T. PATEL & Co., *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 53-61 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

Describes the work carried out in prestressed concrete for a convocation hall at the University of Travancore, Trivandrum, India. Design and construction details are covered, including roof purlins, gantry girders (two spans continuous) for overhead crane, and the diagrid (two-way) prestressed slab.

Transmission length and bond stress in pretensioned prestressed concrete

R. H. EVANS, *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 65-67 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

Deals with the magnitude of the wedge or Poisson's ratio action in pretensioned prestressed concrete and discusses the effect of the surface conditions of the wire on the value of the transmission length. Reference is made to the influence of rust, surface indentations, and crimping. Attention is also devoted to strands or wire ropes and deformed high tensile steel bars. Both types of tendons are discussed and typical results given concerning their bonding characteristics. The advantage of strands and deformed high tensile bars over plain wires, when a prestressing tendon with a short transmission length is required, is emphasized. This is particularly so in members subjected to large bending moments and shearing forces near their ends, such as railway ties and floor joists.

The effect of time or the creep of transmission lengths is examined and representative values given for bright and rusted wires, and for wire ropes. Expressions are presented for calculating the transmission length of wires and wire ropes from the measured pull-in. The nature of bond is finally discussed based on results obtained by an x-ray technique.

Precast prestressed concrete as applied to construction of building stairs

R. N. CHATTERJEE and A. K. BANNERJEE, *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 41-44 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

Describes prestressed concrete stairs consisting of precast prestressed concrete stringer beams and precast reinforced concrete steps suported in between. Plans are presented as well as design calculations and details of manufacture and erection; costs (in India) are compared with cast-in-place stairs.

Stresses in a three-dimensional prestressed concrete frame

R. H. EVANS and J. J. FAFTERY, *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 77-86 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

The tests discussed were designed to supply some practical information and data on the reliability of elastic and plastic design techniques when applied to an open rigid prestressed concrete frame. The frame was 26 ft x 10 ft x 7 ft high and the 26 ft length was di-

vided into two bays of 16 ft and 10 ft.

Up to at least the working load, the elastic theory worked out with reference to the applied distortions, gave an accurate picture of the frame behavior. This relationship would not be true if creep and permanent set were not linearly related to the instantaneous strain within the stress limits used in the investigation.

When analyzing the ultimate strength of the frame, the joints may be treated as fully rigid but within the elastic range, 95 percent rigidty at the joints was an average value and only one exceptionally low value of 40 percent rigidity was recorded. The relationship between the instantaneous load and the deflection is linear up to 50 percent of the crushing strength for increasing deflection but shows the effects of permanent set when reducing the deflection.

After the concrete has cracked the subsequent behavior of the frame can be closely explained by assuming the progressive formation of hinges at the cracked sections.

Analysis of stresses in a prestressed beam using Araldite models

K. D. Mahajan, *Indian Construction News* (Calcutta), V. 7, No. 8, Aug. 1958, pp. 112-117 (from the Proceedings, Symposium on Prestressed Concrete as Applied to Buildings, Central Building Research Institute, Roorkee, Feb. 1958)

Author's Summary

Reports the results of photoelastic investigation to determine: (a) the maximum principal stress; (b) the general stress distribution in the anchorage zone; and (c) the extent to which these local disturbances extend.

Designing of prestressed reinforced concrete construction accepted in USSR

G. I. Berdichevsky, *Indian Construction News Supplement* (Calcutta), V. 8, No. 5, May 1959, pp. 2-10

Since 1958 prestressed structures in the USSR have been designed on the basis of the "limit" method. As set out in the standards, the requirement is stipulated that the load strain or stress shall, at worst, not exceed the ultimate stress of strength of material of the designed construction. Prestressed constructions are designed so that no ultimate conditions are reached in the pretensioning of concrete and throughout the construction's life.

Three principal limiting conditions are accepted for prestresed reinforced concrete construction: (1) the carrying capacity limit is a condition which on attaining the construction either loses its ability to resist external effects and fails or becomes permanently deformed, thus forbidding further exploitation; (2) the general deformations limit, in which calculation of reinforced concrete constructions are performed for standardized loads and standardized values of deformation as allowed; and (3) the cracking forming and crack opening limit is considered in cases and for constructions when the appearance of cracks or their excessive opening precludes any further exploitation or drasticaly detracts from the exploitation properties of the construction.

Prestressed reinforced concrete constructions are divided into three categories of crack resistance, according to the degree of the danger of crack formation: (1) The first category includes constructions which must be impermeable. (2) The second category includes constructions to which the impermeability requirement is not imposed but which are under the effects of at least one of the following factors: an aggressive medium; repeated loads; or high-grade reinforcement. (3) The third category includes constructions reinforced with rolled rod or prestressed members, or by cold-drawn wire used for welded frames and webs.

USSR research in prestressed reinforced concrete is also described.

Fire resistance of prestressed concrete

THOMAS KLUZ, *Concrete and Constructional Engineering* (London), V. 54, No. 7, July 1959, pp. 231-241

Describes a Polish investigation of fire resistance of proposed prestressed floors and roofs. Objects of investigation were to study changes in the elements during heating; determine modulus of elasticity and ultimate strength before and after heating; and determine losses of prestressing caused by heating.

Test results showed that the modulus of elasticity of concrete decreases rapidly at high temperatures; at about 500 C it was found to be less than 1/20 of that at 20 C. After heating to a high temperature the value of the modulus of elasticity of prestressed concrete increases rapidly with cooling but never regains the original value.

Reported losses of prestressing in the 6 ft 6 in. specimens ranged from 10.2 percent for slow cooled specimens heated from one side to 33.8 percent for rapidly cooled specimens heated from both sides.

Design of prestressed suspension roofs

W. ZERNA, *Architectural Science Review* (Sydney), V. 2, No. 2, July 1959, pp. 97-100

Advantages and special problems involved in the design of suspended prestressed concrete roofs are briefly discussed. Essentially, this is a translation of the author's article in *Precontrainte*, No. 2, 1957.

Prestressed concrete in torsion

HENRY J. COWAN, *Constructional Review* (Sydney), V. 32, No. 9, Sept. 1959, pp. 20-25

Prestressed concrete is suggested as the solution to unusual architectural problems involving torsion such as space frames, balcony girders, and helical staircases. Methods of analysis are suggested.

A plastic theory for the ultimate strength of prestressed concrete beams

R. M. RAO, *Indian Concrete Journal* (Bombay), V. 33, No. 9, Sept. 1959, pp. 306-309

In recent years many plastic theories have been proposed for determining the ultimate strength of prestressed concrete beams, the main difference between the various theories being the assumed shape of the stress-distribution curve for concrete above the neutral axis. The problem has been to find a convenient representation of the distribution, which will give accurate predictions of ultimate strength without involving complex mathematics.

Perhaps the most exact theory is that due to Morsch, which uses the actual stress-strain curve as the distribution and locates the position of the resultant compressive force by graphical integration. More recent theories have used a rectangular or trapezoidal stress block to simplify the calculations. The author applies Jensen's trapezoidal stress block theory to develop a plastic theory which has given accurate predictions of the ultimate load of both pretensioned and post-tensioned beams; the latter type may be either the bonded or nonbonded variety. The author develops formulas based on primary compression failure and primary tension failure for the ultimate strength of uniformly prestressed beams and the ultimate strength of eccentrically prestressed beams and works an example to illustrate the application of the formulas.

Long span prestressed concrete girder roof for factory in West Bengal

K. C. IYYA and A. K. BANERJEA, *Indian Concrete Journal* (Bombay), V. 33, No. 9, Sept. 1959, pp. 310-312

Describes the technique of prestressed concrete used in roofing over uninterrupted floor area of 175 x 142 ft. It was obligatory to provide a roof with two decks, with the lower one flush to enable it to carry ducts for air conditioning the interior of the mill. These requirements were found

to be best fulfilled by adopting prestressed concrete girders with reinforced concrete slabs supported on them. The arrangement consists essentially of six prestressed concrete girders, each spanning 142 ft and spaced 25 ft apart. At the two end gables, reinforced concrete beams running continuous over three spans have been provided. The intermediate girders are of T-section with a top flange of 15 ft width. They are 10 ft deep at midspan and 8 ft deep at the supports, the soffit line of the girders being horizontal. The top flanges of the main girders support reinforced concrete slabs 3¼ in. thick and spanning 10 ft. The soffit is supported on the bottom flange of the main girders and consists of a system of secondary reinforced concrete beams spaced at 8 ft 10½ in. centers and spanning 23 ft betwen the bottom flanges of the main girders.

The girders were prestressed by the BBRV system. The cable method was used in which 16 cables with 42 wires of 5-mm diameter in each cable, tensioned from one end only, were inserted into a steel duct of 54-mm diameter. The ends of the cable are enlarged in diameter to form a trumpet so that it can conveniently accommodate the anchor-head which essentially forms the female part of the final anchorage. This is a post-tensioning system in which the wires are cut to designed lengths, cable is locked at the fixed end, and, after tensioning, a lock nut on the free end bears against the bearing plate. The sequence of operation was to cast first the prestressed concrete and then the two bottom slabs on either side of it. When the bottom slab had hardened sufficiently, the formwork for the top slab was erected on top of it and the top slab concreted. The girder was prestressed 28 days later after ensuring that all the top and bottom slabs were capable of rendering the girder sufficiently stable against torsion.

Apparent modulus of elasticity of prestressed concrete beams under different stress levels

W. N. Lofroos and A. M. Ozell, *Journal*, Prestressed Concrete Institute, V. 4, No. 2, Sept. 1959, pp. 23-47

The load-deflection relationship for a prestressed concrete beam is linear up to the cracking load. This indicates that the modulus of elasticity of the beam is constant within this range. If the load - deflection relationship is studied over a period of time, different conditions exist. Here the camber is increasing with respect to time and the prestressing force is decreasing. From these two factors, it can be seen that the apparent modulus of elasticity of this beam is not constant but a function of time.

The primary purpose of this study is to present, from the results of tests, the following factors: (1) the relationship between the beam, apparent modulus of elasticity, the cylinder modulus of elasticity, and the beam modulus of elasticity; (2) the relationship between the total strain in the concrete, the creep, the shrinkage, and time; (3) the relationship between creep and the apparent modulus of elasticity; and (4) the effect of two different stress levels on all the above.

From the test results of the particular specimens used, the following conclusions were reached: (1) The apparent modulus of elasticity decreased with time. This change was definitely a function of creep. (2) The apparent modulus of elasticity of beams prestressed to 0.40 f'_{ci} was at all times higher than the value determined for the beams at 0.25 f'_{ci}. (3) There does not appear to be a definite relationship between the apparent modulus of elasticity and the cylinder properties. (4) The creep strain plus the shrinkage was linear across the cross section. (5) The creep in the concrete at different locations of each beam appeared to be a function of both the stress magnitude and the relative shrinkage. (6) There is no marked

difference in the creep or rate of creep readings taken at corresponding locations in beams prestressed to different stress levels.

Prestressed soda silos at Janikowo

J. LANGOWSKI, *Przeglad Budowlany*, No. 2, 1958, pp. 52-57

Surveys the construction of three silos. These silos were constructed of prefabricated walls, dome plates, and units used for a gallery crowning the silo. After assembling the tanks, they were prestressed by circumferentially wound wire; and finally a coating of concrete, 1¼ in. thick, was added to provide protection to the wires and the wall from rain.

Properties of Concrete

Variation in strength of portland cement concrete

DAVID A. JARMAN, *Civil Engineering and Public Works Review* (London), V. 54, No. 637, July-Aug. 1959, pp. 897-901

Summarizes the factors which govern the strength of a concrete, with a new proposal for a limited working range of water-cement ratios. The author considers only two factors to influence the strength of a particular portland cement concrete. (1) the volume of the water-plus-voids remaining between the solid particles of the concrete mix at the conclusion of casting operations; the progressive filling of these water-plus-voids spaces by cement hydrates constitutes the phenomenon of hardening; and (2) the ability of the cement to provide these hydrates, which is a function of the cement quality and of the quantity of the cement present in the concrete.

Both water content and air voids content have therefore, an important influence on concrete quality, and the water-cement ratio law of Abrams is a special case of a more embracing law which includes the water content and voids content of the mix and also a factor dependent on the cement concentration. Plots various strength versus cement-water ratio curves and discusses results of experimental work as carried on by various individuals and laboratories. Concludes with example that for an ordinary portland cement concrete designed for 28-day strength, the cement-water ratio should be between 1.1 and 2.1 or the water-cement ratio between 0.48 and 0.9 for the cement to attain its greatest potential strength at that age.

An in-place strength test for low-density concrete

IRWIN A. BENJAMIN and GEORGE D. RATLIFF, JR., *ASTM Bulletin*, No. 241, Oct. 1959, pp. 23-26 (including discussion)

The Proctor penetration resistance test can be used as a reliable measure of low-density concrete strength (less than 40 lb per cu ft, oven-dry density). The average of five ¼-in. penetrations with the $\frac{1}{20}$-sq in. needle is recommended. The concrete may be tested at any age from a minimum of about 6 hr and may be tested in place without removing concrete specimens or destroying the slab.

It was established that for the low-density concrete, penetration strength is directly proportional to compressive strength for specimens tested under a wide range of density and curing conditions.

A calibration curve of Proctor strength versus cube strength was developed for vermiculite concrete.

Report on cooperative freezing-and-thawing tests of concrete

Special Report, No. 47, Highway Research Board, 1959, 67 pp.

A program of cooperative freezing-and-thawing tests of concrete specimens was conducted by 13 laboratories using the four ASTM tentative methods of tests. Three concrete mixes were used, involving different aggregates and two different air contents. Large variations in durability were found for the same concrete mix and for tests by

the same method, both within and between laboratories. It is indicated that these were due both to differences in the concrete as prepared in the laboratories and in treatment of the specimens by given test method.

Results of the cooperative tests indicate that significant progress has been made in improving the freezing-and-thawing test methods during the years since the earlier Highway Research Board programs. This is clearly shown by the reduction in coefficient of variation within laboratories to less than ½ of that obtained previously. However, even with the help of statistical procedures applied to results of the present carefully planned program of tests, there remains some latitude for difference of opinion as to the over-all meaning of the data. This is evident in the different sections of the report in which different authors, although dealing with the same test results, place somewhat different interpretations on their significance.

Alkalies in cement and the stability of concrete (in Russian)

G. S. Royak, *Beton i Zhelezobeton* (Moscow), No. 7, July 1959, pp. 295-299
Reviewed by D. Watstein

A study of the mechanism of the alkali-aggregate reaction was recently carried out in the Institute for Concrete and Reinforced Concrete. It was concluded that the hydrate of calcium oxide plays a decisive role in the kinetics and degree of development of the corrosive processes in the concrete. The degree of expansion in the concrete was found to depend on the molecular ratio of R_2O in the cement to the potentially reactive SiO_2 of the aggregate. The development of corrosion processes may be avoided by the addition of active mineral admixtures which form compounds with the calcium hydroxide. A change in the ratio of R_2O to SiO_2 either as a significant reduction or an increase will also aid in localizing the corrosion phenomena.

Cement mortars and concretes with additions of synthetic resin dispersants

Georg Sille and Otto Martini, *Silikat Technik* (Leipzig), V. 9, No. 11, 1958, pp. 506-509
Ceramic Abstracts
May 1959 (Hartenheim)

The flexural strength of cement mortars and concrete is inferior to their compressive strength; it can be improved by additions of polyvinylchloride and polyvinylacetate dispersants. The best addition for optimum bending strength is associated with high shrinkage of the mortar on storage in air and the formation of cracks; with $CaCl_2$ additions, a faster initial set and higher compressive strength may be obtained. With synthetic resin mortars, in contrast to cement mortars, the highest strengths can be obtained in a dry climate, as apparently the gradual elimination of water assures the complete hydration of the cement in the synthetic resin mortar. With increasing amount of dispersant, synthetic resin mortars under water and in moist rooms are affected unfavorably and event partially destroyed. Applications are discussed.

Flexural and compressive strength properties of air-entrained concrete with air-cooled blast-furnace slag aggregate

D. W. Lewis and Fred Hubbard, *Proceedings*, ASTM, V. 58, 1958, pp. 1143-1165 (including discussion)
Authors' Summary

The coarse aggregates were from eight sources having different physical properties. One source having average physical characteristics was selected as the "standard" with which the other slags are compared in 6-bag mixes made with a blend of three air-entraining cements. All mixes were repeated on three different mixing days, and strength tests were conducted in accordance with ASTM methods at ages of 7, 14, 28, and 90 days, and 1 year.

Additional variables of aggregate size, cement content, curing condition,

and use of portland blast-furnace slag cement were studied on mixes with the control or standard aggregate.

The results show no consistent correlation of strength with physical properties of the slag aggregates such as unit weight and abrasion loss. With constant cement content, the variations in aggregate size from ¾ to 2 in. produced essentially the same strengths. The ¾-in. aggregate concrete had lower strength. Changes in cement content from 4 to 8 sacks per cu yd produced greater effects on the compressive strength than on the flexural strengths. Rate of strength gain after 7 days was greater in compression than in flexure, and the compression test is indicated to be the more reproducible.

Moist room and water-immersion curing produced equivalent strengths, but exposure to laboratory air after 24 hr moist curing caused significant strength changes. Although lower in strength at early ages, the portland blast-furnace slag cement was equal to the portland blend after 28 days curing.

Long-time creep and shrinkage tests of plain and reinforced concrete

G. E. Troxell, J. M. Raphael, and R. E. Davis, *Proceedings*, ASTM, V. 58, 1958, pp. 1101-1120

Authors' Summary

A comprehensive investigation of creep and drying shrinkage of concrete has extended over a period of nearly 30 years. Many of the specimens were 4-in. diameter unsealed cylinders loaded in compression to 800 psi at 28 days and thereafter stored in air at 50 percent relative humidity. Both creep and shrinkage continued during the entire period. Eighteen to 35 percent of the total creep during 20 years occurred in the first 2 weeks of loading and 64 to 83 percent within 1 year. Fourteen to 34 percent of the total shrinkage during 20 years occurred in the first 2 weeks of drying and 66 to 85 percent within 1 year. Creep increased with a higher water-cement ratio and with a lower aggregate-cement ratio, but shrinkage was not appreciably affected by either of these variables. Both creep and shrinkage of concrete were greater for Type IV than for Type I cement. The creep for sandstone concrete was more than double that for limestone concrete, and the shrinkage of sandstone concrete was more than double that for quartz concrete.

For ages of loading from 28 to 90 days, the older the specimen at time of loading, the less the creep. For stresses from 300 to 1200 psi, the average creep per unit of stress was slightly greater at the high stresses than at the low stresses but for each increment of higher stress the corresponding increment of creep per unit stress was significantly greater. The moisture conditions of storage had a great effect on both creep and shrinkage; the creep for storage in air at 50 percent relative humidity was about 3 times that for storage in water and the shrinkage at 50 percent relative humidity was about 1.4 times that at 70 percent relative humidity and about 8 times the swelling for storage in water. For specimens having a diameter of 4 to 10 in. under moist storage, the creep decreased as the size of specimen increased. For the range of conditions covered by the investigation, the combined shrinkage or swelling and creep varied from about 1 to 11 times the instantaneous deformations.

Small concrete columns of 2000-psi concrete reinforced with 1.9 and 5 percent of steel, and 4000-psi concrete reinforced with 1.9 percent of steel, were stored in air, with corresponding columns in water, to determine the effect of these conditions on the stresses developed in the concrete and steel while under load. The test results show that stress was transferred from the concrete to the steel, until, in a few cases, the concrete was actually in tension and the steel was stressed to the yield point in compression.

Influence of various factors on sulfate resistance of concretes containing pozzolan

MILOS POLIVKA and ELWOOD H. BROWN, *Proceedings*, ASTM, V. 58, 1958, pp. 1077-1100 (including discussion)

AUTHORS' SUMMARY

Reported are results of a research program undertaken to determine the effects of several types of exposure and periods of curing on the sulfate resistance of concrete containing portland cements of high and low C_3A content with and without pozzolan replacements. The three pozzolans employed with each one of the cements included calcined shale, fly ash, and pumicite. The two sulfate solutions used were a mixture of sodium and magnesium sulfates and a solution of sodium sulfate only. The effects of sulfate exposures were judged by several criteria: loss in weight, change in length, change in compressive strength, and visual observation.

It is concluded that the use of a high-quality pozzolan in concretes containing high C_3A cement will greatly improve their sulfate resistance; that the sulfate resistance of a concrete containing the plain cement of low C_3A content is so high that no improvement through use of pozzolanic replacement is possible; that an extended period of curing beyond 28 days has little or no effect in improving sulfate resistance; that the sodium-magnesium sulfate attacks exposed concrete surfaces, causing surface scaling, whereas the sodium sulfate tends to penetrate into the concrete causing swelling and cracking without scaling of exposed surfaces; and that the loss-in-weight method seems to provide the most positive evaluation of potential sulfate resistance.

Variations in portland cement

STANTON WALKER and DELMAR L. BLOEM, *Proceedings*, ASTM, V. 58, 1958, pp. 1009-1043 (including discussion)

In a study of the cement variable, particularly as represented by different shipments from a given source, both mortar and concrete tests were made. This report gives the average results for the various test conditions.

Among the conclusions were: (1) Shipment - to - shipment variations in strength-producing properties of cements from five commercial sources were large in relation to the variations which could be attributed to testing differences. (2) The variability in strength for all five cement sources was sufficiently great to be of considerable importance in controlling the strength of concrete. (3) There was evidence that differences in strength-producing properties of cements were directly related to their tricalcium silicate content.

On the permeability testing of concrete (Betonin vesitiiviyden tutkimisesta)

JUKKA VUORINEN, *Rakennusinsinoori* (Helsinki), V. 15, No. 7-8, 1959, pp. 127-132

AUTHOR'S SUMMARY

After some introductory remarks on the need for standardized methods in testing the permeability of concrete, a brief description of some test methods is presented. Parallel tests have been carried out with three different methods, two of which are being used in Finland and one in Sweden. Nine concrete mixtures were employed, representing different degrees of permeability. The results of the investigation are presented in tabular form and also graphically. Conclusions drawn from the test results are further supported by theoretical calculations of the coefficient of permeability.

Effects of age and water-cement ratio upon the relation between ultrasonic pulse velocity and compressive strength of concrete

M. F. KAPLAN, *Magazine of Concrete Research* (London), V. 11, No. 32, July 1959, pp. 85-92

AUTHOR'S SUMMARY

Experiments have been made on concretes with varying aggregate-cement and water-cement ratios, with the main object of ascertaining the

effects of changes in age and water-cement ratio on the relation between ultrasonic pulse velocity and the compressive strength of concrete. It was found that the ratio of changes in pulse velocity and compressive strength due to a change in water-cement ratio is not generally the same as that due to a change in age. Because of this, the relation between pulse velocity and compressive strength cannot be expected to be independent of age and water-cement ratio. It appears, however, that, for compressive strengths up to about 4000 psi, the relation may for practical purposes be regarded as being so. For concrete of the same aggregate-cement ratio, it was found that low pulse velocity at an early age predicts low strength at later ages.

Mixing time of concrete—Technological and economic aspects (in Hebrew)

R. SHALON and R. C. REINITZ, *Research Paper* No. 7, Building Research Station, Technion, Israel Institute of Technology (Haifa), 1958, 68 pp.

AUTHORS' SUMMARY

In the first part of this study, the effect of mixing time on the strength and variability of concrete was examined.

Batches of plastic concrete with cement contents of 200 and 300 kg per cu m were tested. It was established that up to 1 min mixing time affects the strength of the concrete and particularly its variabiltiy, while longer mixing produces no significant improvement in either uniformity or strength. The cement contents required, as function of mixing time, for the various types of concrete used in reinforced concrete construction in Israel, are given in tables and diagrams.

In the second part, the methods of concrete production used in various types of construction work in different regions of Israel were studied.

Prediction and measurement of temperatures in concrete structures with particular reference to some recent construction in frozen ground

J. D. GEDDES, *Bulletin* No. 11, Department of Civil Engineering, King's College, University of Durham, Mar. 1958, 43 pp., 2/6d

Considers briefly some of the effects of temperature on concrete, proceeds to describe methods of measurement and prediction, and concludes with a description of a recent investigation in which such procedures were adopted.

Details are given of the concreting of a colliery shaft in water-bearing ground which had been frozen as a construction expedient. The study of the variation of temperature with time in the reinforced concrete shaft lining, 2 ft 9 in. thick and 22 ft internal diameter, conifrmed the method of calculation of temperatures.

Effect of fine aggregate characteristics other than grading on strength and durability of portland cement mortar

JOHN G. DEMPSEY, *Proceedings*, ASTM, V. 58, 1958, pp. 1121-1142 (including discussion)

AUTHORS' SUMMARY

The effect on mortar strength and durability of the characteristics of fine aggregate other than grading is examined through tests of sand-cement mortars made at equal water-cement ratio and containing enough sand so that consistency is the same for all batches.

The investigation is divided into six sections: (1) mechanical analysis and regrading to the test standard; (2) classification under the microscope of each size fraction as to edge, surface, and shape characteristics; the particle shapes were used to approximate the surface area of the sample; (3) preparation of mortar samples in accordance with ASTM C 87 for measuring the mortar making characteristics of sand; (4) computation of thickness of the film of water surrounding each solid particle in the mixture and thickness of the film water and cement slurry sur-

rounding each sand particle; (5) measurement of tensile and compressive strength, volume change after 28 days of wet curing and a subsequent 28 days shrinkage in laboratory air; (6) measurement of durability in terms of slow freezing in air and thawing in water and the computation of a "relative durability factor."

The investigation indicated that the cement factor of a cement-sand mortar varies with the roughness, sharpness, and regularity of the surfaces of a natural sand with a fixed gradation, while strength and durability do not necessarily vary proportionally with the amount of cement.

Some question is raised concerning the relative durability of manufactured sand as compared to the natural product. However, the tests confirm the water requirement, and that the generally accepted shape and surface factors have definite significance in defining sand of superior quality.

Effect of rate of loading on some mechanical properties of concrete

R. H. Evans, *Proceedings*, Conference on Non-Metallic Brittle Materials, 1958, pp. 175-190; discussion, pp. 190-192

Road Abstracts June 1959

Results are given of tests on the properties and behavior of concrete and other building materials subjected to rapid rates of loading. Consideration is given to the differences in values obtained for modulus of elasticity by static and by dynamic tests and to the modulus values of some materials for low stresses.

Delayed screeding of concrete

E. N. Mattison, *Constructional Review* (Sydney), V. 32, No. 7, July 1959, p. 30

Brief report indicating an increase in strength of concrete specimens of from 11 to 42 percent obtained by delaying screeding for 3 hr after placement of the concrete.

Cement content as a factor in the corrosion of reinforcement

Rahel Shalon, *Architectural Science Review* (Sydney), V. 2, No. 2, July 1959, pp. 77-81

From chemical data, author finds that ordinary mortar and concrete are sufficiently alkaline to inhibit corrosion of steel in reinforced concrete in the absence of salts. Further, that cement content has practically no effect on the alkalinity of mortar and concrete. Author concludes that the effect of cement content on rate of rusting of reinforcement should not be overestimated, and that the content generally called for by strength requirements is sufficient to protect the reinforcement, provided the concrete is well graded and of adequate consistency to surround and envelope the reinforcement properly.

Creep and shrinkage in lightweight concrete

Truman R. Jones, Jr., and Teddy J. Hirsch, *Proceedings*, Highway Research Board, V. 38, 1959, pp. 74-89

Authors' Summary

Data are given for a large number of laboratory investigations that have been made on creep and shrinkage in structural quality lightweight aggregate concrete. The aggregates are uncoated expanded shale and a clay produced in Texas by the rotary kiln process. A study is made of the effects of variations in the type of cement and the cement content, the type of aggregate and the aggregate gradation, water content, air content, mixing time, age at time of loading, level of sustained compressive stress, and exposure conditions. A comparison is made between values obtained from laboratory specimens and from a full scale prestressed concrete bridge, having one span of lightweight concrete and one span of sand-gravel concrete. Recommendations are made for applying laboratory data to design conditions.

Investigation of compressive strength of molded cylinders and drilled cores of concrete

W. O. Tynes and Bryant Mather, *Technical Report* No. 6-522, U. S. Army Engineer Waterways Experiment Station, Aug. 1959, 26 pp., $0.50

Authors' Summary

The investigation was made primarily to determine relations between 28-day compressive strengths of 6 x 12-in. cylinders molded from concrete mixtures from which aggregate larger than 1½ in. had been removed and that of 6, 8, and 10 in. diameter cores drilled from test structures containing 3- and 6-in. aggregates and having a height equal to twice the diameter. Results showed that estimates of 28-day compressive strength based on tests of either type sample will not be significantly different; however, the smaller the core diameter, the greater the number of cores needed to produce results of a given precision.

Effect of curing temperature on compressive strength was studied by comparing strengths of field- and laboratory-cured cylinders. Results showed that concrete cured at lower temperatures (field) has a lower compressive strength than that cured under standard (laboratory) conditions.

An appendix describes another similar study which produced similar results.

High speed fatigue tests on small specimens of plain concrete

B. M. Assimacopoulos, Robert F. Warner, and Carl E. Eckberg, Jr., *Journal*, Prestressed Concrete Institute, V. 4, No. 2, Sept. 1959, pp. 53-70

A brief description is given of the method of determining the fatigue strength of prestressed concrete beams based on separate fatigue failure envelopes for the concrete and high strength steel. The importance of fatigue tests on specimens of plain concrete and steel is thus emphasized.

The main portion of the paper describes a series of pilot fatigue tests on 2 x 4-in. cylinders of plain concrete which were conducted at speeds of 9000 and 500 cycles per min. The results of the tests are presented as S-N curves and as fatigue failure envelopes for 500,000, 1,000,000 and 2,000,000 cycles.

The authors feel that the experimental work developed in this paper has resulted in the development of a satisfactory technique for high speed fatigue testing of small concrete cylinders. On the basis of a limited number of test results they make the following conclusions: (1) No effect of rate of loading on fatigue strength was observed. (2) The test data was substantially in agreement with the fatigue envelope for concrete as shown in one diagram. (3) Fatigue failure in the specimens tended to occur in the cement paste and at the surface of bonding between aggregate and paste, rather than in the aggregate itself.

Structural Research

Recent research on deformed reinforcing bars

A. W. Hill and K. Hajnal-Konyi, *The Reinforced Concrete Review* (London), V. 4, No. 1, Mar. 1956, pp. 49-57

Reviewed by William R. Lorman

This is a continuation of the discussion of a previous paper bearing the same title and reviewed in the ACI Journal, V. 30, No. 12, June 1959 (*Proceedings* V. 55), p. 1324. On completion of tests by the Cement and Concrete Association, in progress at the time the previous paper was presented, Hill furnished Hajnal-Konyi the resultant test data for study and analsyis. The intepretation of the CACA information deals with (1) steel stresses at first cracking, (2) maximum crack widths, and (3) preventing corrosion of steel reinforcement in concrete structures.

Redistribution of design bending moments in reinforced concrete continuous beams

Alan H. Mattock, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), May 1959, pp. 35-46

Reviewed by Aron L. Mirsky

Two series of two-span continuous

beams were tested. The first series consisted of four specimens designed for the same working load but with various distributions of design bending moment, while the second series, of three beams, simulating secondary beams in a reinforced concrete frame structure, had one beam designed for the bending moment predicted by elastic theory and two (one reinforced with mild steel and one with work-hardened square twisted steel) designed for a 25 percent reduction in negative moment at the support with commensurate increase in positive moments in the spans. It was found that arbitrary redistribution of design bending moments up to 25 percent does not adversely affect the performance of the beams, either in the working-load range or at failure.

See also author's paper, "Limit Design for Structural Concrete," *Journal*, Research and Development Laboratories, Portland Cement Association, V. 1, No. 2, May 1959, pp. 14-24.

Effect of structural reinforcing on vibrational behaviors of a fire-damaged reinforced concrete building

TOSHIHIKO HISADA, KYOJI NAKAGAWA, and SEIJI TAKENOUCHI, *Proceedings of Symposium on the Failure and Defects of Bridges and Structures*, Japan Society for the Promotion of Science, Tokyo, Dec. 1958, pp. 80-89

AUTHORS' SUMMARY

A five-story reinforced concrete building damaged by fire was repaired and at the same time the building was reinforced with earthquake resistant diaphragms (walls). Forced vibration tests were carried out before and after the reconstruction to investigate its effects on dynamic characteristics of the structure.

Comparison of the behavior of concrete beams under static and dynamic loading

J. J. TROTT and E. N. FOX, *Magazine of Concrete Research* (London), V. 11, No. 31, Mar. 1959, pp. 15-24

HIGHWAY RESEARCH ABSTRACTS Sept. 1959

Describes preliminary experiments to determine whether or not the rate of loading significantly affects the mechanical properties of concrete beams tested in flexure. Two rates of loading were used: 5×10^{-7} in. per in. per sec, the rate employed in normal mechanical tests; about 6×10^{-2} in. per in. per sec when the beams were tested under impact conditions.

This change of the rate of loading produced the following results: The load of fracture rose from static values of 749 and 792 lb to dynamic values of 990, 1050, and 1115 lb; the strain at fracture rose from static values of 185 and 243 $\times 10^{-6}$ in. per in. to 200, 287, and 293 $\times 10^{-6}$ in. per in. under impact conditions.

The approximately linear portion of the load-strain line became curved at dynamic strains of 130 and 180 $\times 10^{-6}$ per in. compared with static values of 69 and 85 $\times 10^{-6}$ in. per in.—the corresponding loads were 720 and 900 lb compared with static values of 395 and 530 lb.

If the onset of cracking is associated with the departure from linearity of the stress-strain curve, it appears that increase in the rate of loading markedly affects the strain at which cracking begins.

Relation between the strains in the concrete and the steel in reinforced and prestressed concrete beams

R. H. EVANS and ALAN WILLIAMS, *Magazine of Concrete Research* (London), V. 11, No. 32, July 1959, pp. 55-64

AUTHORS' SUMMARY

Information, obtained by x-ray photography, is presented on the strains in the steel and concrete and on the slip occurring in concrete beams reinforced or prestressed with various types of bar and wire.

An expression is derived relating strain in the steel, virtual strain in the concrete, and slip at the steel-concrete interface. A comparison is made between strains in the concrete at the interface and at the surface. It is found that, for reinforced concrete beams, the strain at the surface is lower than the strain in the interior.

The strain in the steel at the section of failure is compared with the average surface strain in the concrete at the level of the steel. The influence of several factors on this ratio is discussed. It is found that the ratio of the strain in the steel to the average strain in the concrete has a value greater than unity for reinforcement with good bond characteristics and that the ratio increases as loading increases and when the section of failure occurs at a peak in the bending moment diagram.

General

Temperature measurements with thermistors in concrete

KNUD E. C. NIELSEN, *Bulletin* No. 34, Swedish Cement and Concrete Research Institute at the Royal Institute of Technology (Stockholm), 1959, 44 pp., 8Kr.

AUTHOR'S SUMMARY

Deals with the uses of thermistors (thermally sensitive resistors) for temperature measurements in the interior of concrete as well as for temperature measurements, automatic temperature recording, and automatic temperature control in laboratory equipment. The general properties of thermistors and the accuracy in measurements are studied. The stability is the subject of a special investigation. The practical design of thermometers and accessory measuring equipment is described. Four examples of application of thermistors at the Swedish Cement and Concrete Research Institute are given.

Concreting plant, machinery and tools

E. U. BROADBENT, *The Reinforced Concrete Review* (London), V. 4, No. 1, Mar. 1956, pp. 41-48

Reviewed by WILLIAM R. LORMAN

This contribution pertains to the comparative advantages of various components of a small-size central weigh-batching and mixing plant. The author stresses the need for accurate water metering systems and advocates the use of a high discharge-rate type of concrete mixer. The article ends with a discussion of the relative merits of various finishing machines.

National fire codes. V. III—Building construction and equipment

National Fire Protection Association, Boston, 1959, 784 pp., $7.

The National Fire Codes are compilations of standards devloped by the National Fire Protection Association. In most cases the form of the standards is such as to make them suitable only for advisory use but some of the standards have been prepared in the form of model ordinances or have been otherwise arranged to make them suitable for regulatory purposes. Each standard deals with a single feature or a group of related features of the over-all fire protection problem. These standards are intended as a guide and aid in the solution of problems in fire safety providing recommendations to be applied with some judgment rather than as arbitrary rules. The standards themselves do not generally explain the reasoning on which their provisions are based or the actual fire experience which has demonstrated the need for various protective measures.

This volume presents 33 NFPA standards in the field of building construction and equipment. It is not a building code though much of the material herein will be found useful in connection with the drafting and application of building codes. The standards should be applied with due recognition of the fact that safe building construction and equipment *per se* may not be the complete answer to any individual fire problem and that the control of fire hazards, extinguishing and alarm equipment, and other features covered in other volumes of the National Fire Codes should also receive consideration. Model building codes are discussed in an introductory section, before specific items are covered.

ACI NEWS LETTER

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12th Regional Meeting Report—

Varied Program Featured in Mexico City

ALTHOUGH THE HISTORY of the past was evident, the technologies of today and tomorrow predominated at the ACI 12th Regional Meeting in Mexico City, Nov. 2-5, 1959. The meeting attracted more than 350 registrants from Mexico, South and Central America, and the United States.

Mexico City has an impressive history preserved in its architecture. Impressive skyscrapers have shot up next to ancient churches, monasteries, and colonial palaces. Wide boulevards intermingle with narrow, winding streets. It was in this busy up-to-date metropolis, that preserves its old world flavor, that the ACI 12th Regional Meeting reviewed recent concrete construction progress and previewed the structures of tomorrow.

The technical sessions were held in the College of Engineering building at Ciudad Universitaria (University City), while various tours took ACI members as far afield as Xochimilco and Cuernavaca.

University City, the pride of modern Mexico, is probably the largest university center in the world and also one of the most beautiful. It is set in Pedregal, an immense lava flow that covered the valley of Mexico thousands of years ago. This setting, with its backdrop of modern architecture and structural engineering, proved ideal for a program that included speakers from Mexico, Cuba, United States, and Germany.

PROGRAM SUMMARY

A full 3 days were devoted to technical sessions covering a wide range of subjects. Design and construction of concrete shells, as well as research, received attention. Various aspects of prestressed concrete were covered, including European construction, bridges, arches, pipe, power poles, and railroad ties. Deep beams and shear investigations of beams and frames without web reinforcement were discussed. Shear walls in tall buildings and folded plate raft foundations were other topics.

The relationship between research, building codes, and design practice was presented. Concrete technology in water resources development was reviewed. One complete session was devoted to reinforcing steel, particularly as it applied to Mexican construction. Other papers were devoted to building with precast thin shell panels, concrete dams and foundation grouting, ready-mixed concrete in Mexico, and the alkali-aggregate reaction.

"Extra curricular" in a sense, but still an inherent part of the technical program, were the luncheons and tours during the meeting. These included an inspection of the facilities of University City, with particular emphasis on the Institute of Engineering facilities and laboratory. There was also a visit and picnic luncheon at the Xochimilco floating gardens and a visit and luncheon at the new Bacardi plant now under construction about 20 miles outside Mexico City. After the technical program, various tours were available to registrants on Friday, November 6, which gave visitors an opportunity to inspect design and construction features on various types of structures and foundations.

Firms supplying materials, equipment, and services used in various fields of civil engineering also participated in an exhibition with displays of products in the Engineering Building at University City.

ADDED ATTRACTIONS

In addition to the luncheons and tours, there was the welcoming social hour on Monday evening, November 2, and the dinner dance on Thursday evening, November 5.

While the technical sessions were in progress, the local committee also organized an interesting and busy schedule for the ladies. This included a visit to the National Palace and National Museum of Mexico; a tour of shopping centers in down-town Mexico City and typical silver and leather shops; and the Nacional Monte de Piedad (Charity Pawnbroker Institution). The ladies also visited houses in the Village of Obregon and saw the Tenayuca Pyramid on the outskirts of Mexico City. In addition to luncheons at University City and Xochimilco floating gardens, the ladies breakfasted one day at the Terrace of Chapultepec Castle and visited the museum there. Visits were also arranged to the Museum of Religious Art, the Metropolitan Cathedral, and the Shrine of Guadalupe.

ORGANIZING COMMITTEE

Alberto Dovali Jaime, consulting engineer, was general chairman of the organizing committee with Adolfo Zeevaert, consulting engineer, serving as secretary. Antonio Robles, Jr., Cemento Mixcoac, served as treasurer and Luis Flores Arias, civil engineer, was in charge of finances.

The technical program was under the direction of Leonardo Zeevaert, consulting engineer, and technical inspection trips were under the direction of architect Felix Candela.

Credit is also due to Francisco Santos Oliva, consulting engineer, educational contacts; Enrique Torres de Alba, civil engineer, publicity; Alfonso Marin E., civil engineer, membership promotion; A. Pardo Bolland, civil engineer, exhibits and displays; Rafael Hernandez y Hernandez, civil engineer, social events; and Mrs. Marcela Castro Villagrana de Hernandez, ladies' program.

TECHNICAL PROGRAM

The technical sessions were opened officially on Tuesday morning, November 3, by Alberto Dovali Jaime.

The papers, presented in either Spanish or English, are summarized briefly in this NEWS LETTER. It is anticipated that many of these will be submitted for possible publication in the ACI JOURNAL and that many will also be published in Spanish in *Ingenieria*, the official publication of Escuela Nacional de Ingenieria, Universidad Nacional Autonoma de Mexico.

Stress analysis of the hyperbolic paraboloid treated as a translational shell (Analisis de esfuerzos en el parboloide hiperbolico considerado como superficie de translacion)

Jorge Molina Montes and Ignacio Canals N.

A hyperbolic paraboloid was treated as a translational surface, permitting an easier evaluation and determination of the functions determined by the boundary conditions. Based on the skin theory developed by Pucher, formulas were derived for obtaining stresses acting on the surface. These were compared with those obtained by other authors (Aimond, Candela, and others). Border conditions were briefly reviewed. A practical example was presented.

The velaria, its analysis and design (La velaria, su analisis y diseno)

Ignacio Canals N. and Guillermo Gargollo

The "Velaria" (warped surface), which can be defined as the surface that a membrane adopts under external forces, was treated briefly. The equation for such a surface limited by stiff members forming a rectangle and under a uniform distributed load was derived. The stresses on the membrane were analyzed and a study was made of the variation of these stresses, as the surface varies, due to slope of the membrane at the borders, etc. Formulas for the computation of these stresses were presented and several numerical examples reviewed.

Deep beams (Trabes aperlatadas)

Elihu Geer, professor and chairman, Department of Civil Engineering, University of Detroit, Detroit

The deep beam theory is in contrast with the usual flexural theory in that it is mainly one of distributing a localized force. One unusual feature of deep beams, discovered in the course of this investigation, was that the greatest tensile stress occurs not at the midspan but near the face of the support. Another interesting feature was that maximum stresses are a function of the magnitude of the load and not its location, e.g., the stresses caused by a load at the center of the beam are almost of the same magnitude as when the load is placed near the support. This study was limited to deep beams whose height was at least 0.5 of its clear span. (This paper was published in the January 1960 ACI JOURNAL.)

"Dywidag" prestressed concrete (Concreto preesforzado "Dywidag")

Ulrich Finsterwalder, technical director, Dyckerhoff & Widmann K. G., Munich, Germany

Various prestressed concrete structures were described and illustrated. The projects included bridges, auditoriums, water towers, and also covered such details as cast-in-place and precast construction, prestressing technique, materials control, and some design and construction analyses and details.

Prestressed concrete bridge construction in Mexico (La construccion de puentes de concreto preesforzado en Mexico)

Vicente Guerrero y Gama

Probably one of the main factors encouraging the design of prestressed highway and railway bridges in Mexico is the shortage of structural steel and the high relative cost. About 35 percent of the structural steel consumed in Mexico is imported.

The first prestressed concrete bridge in Mexico was built in 1952, with a total length of 175 m. From 1956 to date, prestressed concrete bridges have been built at an increasing rate and, at

the same time, steel bridges have practically disappeared on highway projects. In fact, for spans longer than 25 m even the use of reinforced concrete has decreased in favor of prestressed construction.

Elastic analysis of shear walls in tall buildings (Analisis elastico de muros rigidos en edificios altos)

Emilio Rosenblueth and Ignacio Holtz, respectively, director, Institute of Engineering, University of Mexico, and structural designer, Diseno Racional, A. C. (DIRAC), Mexico City

Often the moment-resisting frame of a tall building is supplemented by shear walls running the full height of the building. Under lateral loads, interaction between the shear walls and frames, due to flexural deformations of the walls, leads to complex analytical problems. These are amenable to an approach by successive approximations similar to that of beams on elastic foundations. If the building is fairly regular a good first approximation results from the solution of a differential equation based on the assumption of complete uniformity. The solution was given as a set of graphs. The authors applied the method to one particular example and found that the first approximation was quite good. A numerical method was set up to systematize successive cycles of iteration and it was found that extrapolation from the first two cycles supplied almost exact results.

Prestressed precast concrete arches span 102 ft 6 in. for industrial roof (Arcos de concreto precolados y preesforzados de 31.24 m de claro para un techo industrial)

E. R. Cancio and A. Herrera, Saenz-Cancio-Martin-Gutierrez, Ingenieros-Arquitectos, Havana, Cuba

The solution adopted for the structural design of an industrial building in Havana, to meet special manufacturing and esthetic requirements, was described. The solution consisted of prestressed joints and tied arches having spans up to 102 ft 6 in. Prefabrication was used to a large extent. The prefabricated, two-hinged tied arches were described in detail. Unit costs were also given.

Prestressed pipes and prestressed power poles (Tubos y postes preesforzados)

R. J. Sweitzer, director of research, Lock Joint Pipe Co., Wharton, N.J. (Presented by Mr. Tiscareno, Lock Joint Pipe Co. de Mexico)

Manufacturing techniques and applications were described for both prestressed pipe and prestressed power poles. Details were given on tests on the prestressed concrete poles and tests on prestressed concrete embedded cylinder pipe were summarized. The poles were of lattice design and can be produced in lengths up to 50 ft.

Multi-purpose concrete building built of precast thin-shell panels (Edificio de concreto para usos multiples construido de cascarones en tableros precolados)

Arsham Amirikian, special structures consultant, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

An assembly of simple precast panels, connected together by bolts and welded inserts, provides the structural framing of a single-story gable-roof building, suitable for many uses. The panels are ribbed type, consisting of a thin slab and peripheral stiffening members. Arranged for 4-ft modular framing, two pairs of panels—one pair placed in the roof and the other in the sidewalls—compose a building segment 4 ft wide.

Currently the concept is being utilized in the development of a series of framings for a large-scale project. Incidental to this application, some problems of framing arrangement, design, assembly, and jointing were discussed. Information was also given on strength tests and service behavior of an experimental building of these panels.

Hyperbolic paraboloidal umbrella shells under vertical loads (Cascarones paraboloide hiperbolicos tipo sombrilla, de concreto reforzado)

Howard P. Harrenstien, assistant professor of civil engineering, Iowa State University, Ames

Tests were performed on two reinforced concrete hyperbolic paraboloidal umbrella shells. The shells were sub-

jected to a concentrated load at various selected points, and resulting strains were observed. Stresses corresponding to these strains were determined, and the results were presented in the form of stress contours for each of nine selected load points. Superposition methods were presented which predict the final stresses in an appropriate umbrella shell when it is subjected to any vertical loading provided the stresses remain within the proportional limit, and provided that deflections are small.

Folded plate raft foundation for 24-story building (Diseno de una trabe—Losa corrida de concreto para cimentacion de un edificio)

Ignacio Martin and Sixto Ruiz, Saenz-Cancio-Martin-Gutierrez, Ingenieros-Arquitectos, Havana, Cuba

The folded plate raft foundation of a 24-story building 308 ft high was described. The influence of the subsoil conditions and the wind load were discussed. Costs were compared for a standard beam and slab raft foundation and the folded plate raft foundation. (This paper was published in the August 1959 ACI JOURNAL.)

Behavior and strength in shear of reinforced concrete beams and frames without web reinforcement (Comportamiento y resistencia al esfuerzo cortante de vigas y marcos rigidos de concreto sin refuerzo en el alma)

Roger Diaz de Cossio Instituto de Ingenieria, Universidad Nacional Autonoma de Mexico, Mexico City, and Chester P. Siess, professor of civil engineering, University of Illinois, Urbana

Forty-nine simply-supported beams and 24 frame members were tested under different combinations of loading. These included: two-point loads, midspan concentrated loads, uniform load, and axial load. The frames and 18 simply-supported beams were tested under uniform load. Ten of the beams under midspan concentrated loads were tested with an axial load of 20 kips. Other variables were concrete strength, steel percentage, and span length. The tests were discussed and correlated. The use of the inclined tension cracking load as a measure of the useful shear strength of the members was advocated, and a procedure given for its determination. (The paper is published in this month's JOURNAL.)

General formulas for membrane stresses in hyperbolic paraboloidical shells (Formulas generales para esfuerzos de membrana en cascarones formados por paraboloides hiperbolicos)

Felix Candela, architect, Mexico City

This general set of formulas gave the membrane stresses in a hyperbolic paraboloid loaded with its own weight but disposed in an arbitrary position in space, that is, when the load has components along the three axes of the hyperbolic paraboloid (hypar). The better known formulas for the hypar with its z axis in a vertical position can be obtained as particular cases of this more general set of equations.

Research, building codes, and engineering practice (Relacion entre investigacion y especificaciones y el efecto de estos en la practica de diseno)

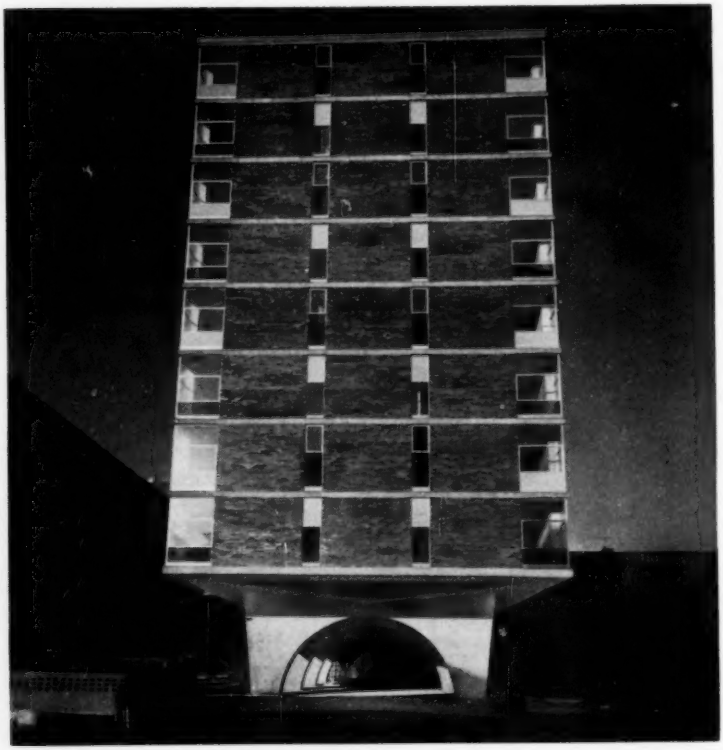
Chester P. Siess, professor of civil engineering, University of Illinois, Urbana

The function of research and practice as sources of knowledge and the utilization of this knowledge in the preparation of codes or specifications were considered from a philosophical and historical point of a view. Examples drawn from current or past ACI Building Codes were cited to illustrate the roles played by research and by engineering practice in the drafting or revision of codes.

Mexican high strengths steels for concrete reinforcement (Aceros Mexicanos de alta resistencia como refuerzo del concreto)

Luis Gimenez Cacho, manager, Barras Performados, S. A., Mexico City

The development and introduction of high strength steels in Mexico was commented on and the characteristics of cold twisted versus hot rolled steels was discussed. Several types of deformations in use in Mexico were described and the term "unitary cross sectional area" was introduced as a



Reinforced CONCRETE Arches

Support Luxury Apartment Building

Proving that you can do the unusual with reinforced concrete is this luxury apartment building designed by Tasso Katselas. In this design, the entire first floor is supported directly on reinforced concrete arches which serve as an open colonnade leading to stairways and elevators. At ground level, the arches also support reinforced concrete columns extending the full height of the building, from which flat slab floors with continuous cantilevers extend around each side of the structure.

Only with reinforced concrete can you achieve such freedom of design. Before you build any type of building, investigate this more flexible medium of construction.

Neville House Apartments,
Pittsburgh, Pa.
Architect: Tasso Katselas, Pittsburgh, Pa.
Structural Engineers: Gensert, Williams & Associates, Cleveland, Ohio
Contractor: Gratziano Construction Company, Pittsburgh, Pa.
Photos by Jay-Bee Studios

Concrete Reinforcing Steel Institute
38 South Dearborn St., Chicago 3, Ill.

basis for comparison of deformations. Main advantages of the use of high strength steels were indicated as well as some of the limitations.

Special reinforcing steel for concrete (Aceros de refuerzo para concreto)

Cia. Fundidora de Fierro y Acero de Monterrey, S. A.

Reinforcing steel as developed and used in Mexico was discussed in detail. Particular attention was paid to manufacturing processes and techniques.

Cold twisted steel produced in Mexico (El acero torcido en frio que se produce en Mexico)

Max Calva Cuadrilla, Aceros Ecatepec, S. A.

Material specifications, fabrication, and research on cold twisted steel were discussed. The physical characteristics of the steel wire explained and design tables from different countries. for this type of steel, were illustrated.

Recent developments in cement grouting used in the construction of dams (Recientes adelantos en el inyectado con cemento aplicado a la cimentacion y construccion de las presas)

A. Warren Simonds, consulting engineer, Denver

A specific example of the modern practices of cement grouting, the recently completed grouting program at Monticello Dam, was described.

Pressure grouting is the process of injecting suitable cementitious slurries or similar materials into inaccessible places such as the underlying formations of foundations of structures for the purpose of sealing seams, cracks, and fissures, or filling voids. While the principal use of this process is to fill openings in a structural mass and render it impervious to percolating water, it is used also to improve the strength and elastic properties of the material into which it is injected. This process has been used extensively in the construction of a variety of structures such as dams, buildings, and tunnels.

Grouting to improve the foundations of dams in Mexico (Inyectado para el mejoramiento de la cimentacion de las presas en Mexico)

Aurelio Benassini and Federico Barona de la O, respectively, chief engineer and materials engineer, Ministry of Hydraulic Resources, Mexico City

Information given referred to the grouting techniques used in the construction of some important dams. Data on Mexican dams were compared with some of the dams built abroad. Some of the different problems encountered in some of the dams built in Mexico were discussed, specially in the case of El Marques Dam where the fissured limestone formation constituted a difficult problem that was solved by the use of some special techniques. Reference was also made to five other concrete dams that presented technical features of special interest.

Ready-mixed concrete (Concreto premezclado)

Livio Pontecorvo

Statistics were given on the production of ready-mixed concrete in Mexico and plant operations were described.

Rigid prestressed concrete railroad tie for Mexico (Un durmiente de concreto precomprimido tipo rigido para Mexico)

Alberto Dovali Jaime, consulting engineer, Mexico City

This paper discussed Mexico's railroad tie supply problem. The government is studying the possibility of using concrete ties rather than wood. A type of tie used in Germany was described and details were given on manufacture of the units as well as the methods of attaching railroad rails to the ties.

Four decades of improved concrete technology for water resource development (Cuatro decadas de tecnologia mejorada de concreto para el desarrollo del aprovechamiento de recursos hidraulicos)

Grant Bloodgood, assistant commissioner and chief engineer, Bureau of Reclamation, United States Department of the Interior, Denver

The work of the U. S. Bureau of Reclamation was used to illustrate developments in concrete technology over the past four decades. While many of the new ideas have been developed through laboratory research, it has taken great pioneering effort by design

and construction engineers to put them into practice. Developments spotlighted included water-cement ratio, screening of aggregate, batching by weight, control and inspection, low heat cement, equipment, control of alkali-aggregate reaction, use of pozzolans, air entrainment, use of admixtures, and others.

Alkali-aggregate reaction in concrete (Reaccion alcalis-aggregabos en el concreto)

Bryant Mather and George Verbeck, respectively, chief, Special Investigations Branch, Concrete Division, U. S. Army Engineer Waterways Experiment Station, Jackson, Miss., and manager, Applied Research Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

A review of the alkali-aggregate reaction problem—the research that has been done and is in progress and how this knowledge is being used in the production of concrete.

It has now become apparent that, in order for the particular sort of trouble that is caused by "alkali-aggregate reaction" to occur, it is necessary that three conditions be met: (1) the aggregate must consist of or contain significant quantities of materials that are composed of silica in one or more of its less stable forms; (2) the cement must contain, or the concrete must acquire from sources other than the cement, a significant quantity of the alkalines, sodium and posassium;(3) the concrete must be subject to wetting. If any one of these three conditions is not met, excessive expansion and cracking due to alkali-aggregate reaction will not be expected to occur and no special precautions need to be taken to prevent their occurrence.

The discussion covered both geological and chemical aspects of the problem. Various ways of dealing with the problem were outlined.

The prestressed German tie and its success (El durmiente Aleman de concreto precomprimido y su exito)

Hermann Meier, Deutsche Bundesbahn, Minden, Germany

The various concrete railroad ties, both reinforced and prestressed, produced and used in Europe were described. The evolution of these units and their development, based on research and field experience, was outlined. Manufacturing techniques were described. Various methods of attaching rails to the ties were described; regulations of various European countries, regarding shape, dimensions, reinforcement, were also mentioned.

Closing comments

Leonardo Zeevaert then summarized the various papers and the meeting was officially closed by the chairman, Alberto Dovali Jaime.

1960 Regional Meeting

The 13th ACI Regional Meeting is scheduled for Oct. 31, Nov. 1-2, 1960, in Tucson, Ariz. Gene M. Nordby, head of the department of civil engineering, University of Arizona, will act as general chairman of the meeting.

PINTURAS PARA HORMIGON

Spanish translation of two ACI reports on the application of paint to concrete. Prepared from ACI Standard 616-49, **"The Application of Portland Cement Paint to Concrete Surfaces,"** and the Committee 616 report on **"Guide for Painting Concrete."***

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ACI PUBLICATIONS DEPARTMENT

*Both reports are available as separate booklets in English.

POZZOLITH . . . makes good concrete better

ALLEN STEAM PLANT project at Belmont, N.C. is part of Duke Power Company's continuing expansion program to meet growing power needs in the Piedmont area. Project includes approximately 73,000 cubic yards of POZZOLITH concrete.

POZZOLITH concrete specified again for Duke Power Company's continuing $200 million expansion

Duke Power Company—with the largest privately-owned, steam-operated electric capacity in the southeast—handles its own design, engineering, construction and concrete production. These operations are directed by C. T. Wanzer, Chief Engineer —C. E. Watkins, Construction Manager and R. L. Dick, Resident Engineer.

On the Allen Steam Plant project—several concrete mixes were designed with POZZOLITH and Duke Power Company's fly ash. The mix that best met job requirements most economically was:

Cement— 353 lbs.
Fly Ash (Duke Power)— 117 lbs.
Sand—1215 lbs.
1″ Crushed Stone—2057 lbs.
Total Water— 30 gal.
POZZOLITH

The job concrete was easily pumped more than 600 feet through an 8-inch line from the automatic batching plant. Slump of 2″ to 3″ was maintained—there was excellent workability and cohesiveness . . . and high, uniform strength . . . 2250 psi at 7 days . . . 3700 psi at 28 days . . . and 4900 psi at 90 days. The specified 28-day strength of 3000 psi was easily exceeded.

The local Master Builders field man will welcome discussing your requirements. Remember, POZZOLITH concrete is higher in quality and more economical than plain concrete or concrete made with any other admixture.

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Welcome to New York City

56th Annual ACI Meeting

■ NEW YORK CITY—the convention and visitors capital of the world—will play host to ACI members and guests at the 56th annual meeting of the Institute, March 14-17, at the Commodore Hotel.

The Technical Activities Committee has arranged a program geared to the interest of the designer, builder, research man, and others in the fields related to the concrete industry. Eminent concrete authorities will present about 30 papers at the eight scheduled sessions.

Technical committee "working sessions" will take up the first 1½ days of the convention program. Committee reports will be presented at the first general session on Tuesday afternoon, March 15. On Wednesday morning, March 16, the technical sessions will begin with two sessions running concurrently through Thursday noon. The final research session is scheduled for Thursday afternoon.

The local planning committee with Roger H. Corbetta, president of Corbetta Construction Co., New York, as general chairman, has arranged a number of "extras" to make the 56th annual meeting one of the finest in Institute history. "Milady" has been given prime consideration in special events planning and will be pleasantly occupied while ACI'ers are attending technical sessions. Technical program for the meeting is outlined below.

TUESDAY AFTERNOON, MARCH 15

First General Session

Committee Reports:

Committee 326, Shear and Diagonal Tension
Committee 401, Specification for Structural Concrete
Committee 609, Consolidation of Concrete
Committee 622, Formwork for Concrete

WEDNESDAY MORNING, MARCH 16

Model Test Session

Structural Models Evaluate Behavior of Concrete Dams
Jerome M. Raphael, University of California Berkeley

Glen Canyon Dam Structural Model Tests
George C. Rouse, U.S. Bureau of Reclamation, Denver

Experiments with Thin-Shell Structural Models
J. L. Waling and Longin B. Greszczuk, Purdue University, Lafayette, Ind.

Testing Rigid Frame Bridge Model to Ultimate Load
C. S. Wu, California Institute of Technology, Pasadena,

A. A. Pap and D. H. Pletta, Virginia Polytechnic Institute, Blacksburg

Highways Session

Air-Entrained Concrete After 20 Years
E. A. Finney, Michigan State Highway Department, East Lansing

Continuous Placement of Concrete on Three-Span Continuous Units of Bruckner Expressway
H. B. Britton, State of New York Department of Public Works, Albany

The AASHO Road Test—A Background Report
Frank H. Scrivner, AASHO Road Test, Ottawa, Ill.

Gravel Beneficiation in Michigan
F. E. Legg, Jr., University of Michigan, Ann Arbor, and

W. W. McLaughlin, Michigan State Highway Department, East Lansing

WEDNESDAY AFTERNOON, MARCH 16

Materials Session

Effects of Incomplete Consolidation on Pulse Velocity, Modulus of Elasticity, and Compressive and Flexural Strength of Concrete
M. F. Kaplan, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria

Freeze-Thaw Resistance of Concrete Made with Lightweight Aggregates
Paul Klieger, Portland Cement Association, Skokie, Ill.

Effect of Maximum Size of Coarse Aggregates on Properties of Concrete
Stanton Walker and D. L. Bloem, National Ready Mixed Concrete Association, Washington, D. C.

Concrete-Making Properties of a Portland Blast Furnace Cement Made with High-Magnesia Blast-Furnace Slag
N. Stutterheim, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria

Design and Structural Research Session

Ultimate Strength Design
A. L. Parme, Portland Cement Association, Chicago

Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams
J. G. MacGregor, M. A. Sozen, and C. P. Siess, University of Illinois, Urbana

High Strength Reinforcement in Concrete Structures; Preliminary Report of Committee 339
Robert Sailer, U. S. Bureau of Reclamation, Denver, committee chairman

Shrinkage and Creep of Concrete
Inge Lyse, Norway Institute of Technology, Trondheim

THURSDAY MORNING, MARCH 17

Restoration of Deteriorated Concrete—Committee 201 Symposium

Conventional Methods of Repairing Concrete
Lewis H. Tuthill, California State Department of Water Resources, Sacramento

Repair of Concrete Pavements
Earl J. Felt, Portland Cement Association, Chicago

The Prepacked Concrete Method
Raymond E. Davis, Consulting Engineer, Berkeley, Calif.

Use of Epoxy Resins
Bailey Tremper, California Division of Highways, Sacramento

Pneumatically Applied Mortar
O. N. Kulberg, Southern California Edison Co., Los Angeles

Design and Construction Session

$19 Million Pier for New York City
David P. Billington, Roberts and Schaefer Co., Inc., New York

Effect of Floor Concrete Strength on Column Strength
Albert C. Bianchini, Robert E. Woods, and Clyde E. Kesler, University of Illinois, Urbana

Welding of Reinforcing Steel Between Precast Concrete Units
J. Neils Thompson, Hudson Matlock, and A. A. Toprac, University of Texas, Austin

Shear Strength of Restrained Concrete Beams Without Web Reinforcement
John E. Bower, U. S. Steel Corp., and I. M. Viest, AASHO Road Test, Ottawa, Ill.

THURSDAY AFTERNOON, MARCH 17

Research Session—Committee 115, Research

This session differs from general convention sessions in that the information is not released for publication. A group of short papers will be presented on research techniques and preliminary results on a number of current projects not yet sufficiently advanced to justify final reports.

ACI to cosponsor sessions at ASCE New Orleans convention

ACI, American Society of Civil Engineers, and Prestressed Concrete Institute will cosponsor five sessions on prestressed concrete in connection with the ASCE New Orleans convention, March 7-11 at the Jung Hotel.

ACI President Phil Ferguson, University of Texas, Austin, will serve as chairman of the experimental research session and ACI member Philip E. Balcomb, Concrete Products of America, Division of American-Marietta Co., will act as chairman of the construction projects session. The other three scheduled sessions will be devoted to buildings and structures, bridges, and recent developments. Approximately 20 papers will be presented at the sessions.

ACI publishes Spanish translation of paint reports

The American Concrete Institute has published a Spanish translation of two ACI reports on applying paint on concrete. It is a translation of two current ACI publications: "Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)" and the Committee 616 report, "Guide for Painting Concrete." The 39-page bulletin was translated by the Instituto del Cemento Portland Argentina, Buenos Aires. It is available from ACI headquarters in Detroit at $1 per copy.

Part 1 of the booklet describes types of paints, other than portland cement paint, commonly used on concrete. It reviews procedures for preparing the surface, selecting and applying the paint, and for repainting. Dampproofing and waterproofing are discussed briefly.

Part 2 translates the ACI Standard ACI 616-49 setting forth recommended practice for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

Steinman takes associates into firm partnership

D. B. Steinman, consulting engineer, New York, has taken into partnership his associates R. M. Boynton, C. H. Gronquist, and J. London. These associates have been with Dr. Steinman from 31 to 38 years, and have been identified with him in the design and construction of over 400 bridges including some of the world's most notable structures.

The new firm will operate under the name of Steinman, Boynton, Gronquist and London, Consulting Engineers.

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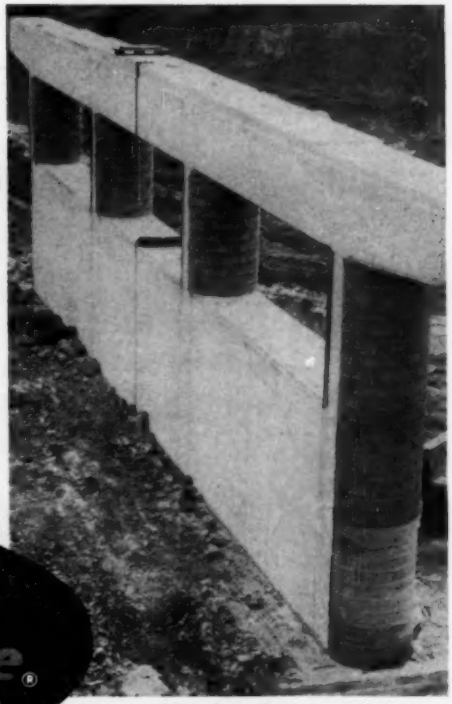

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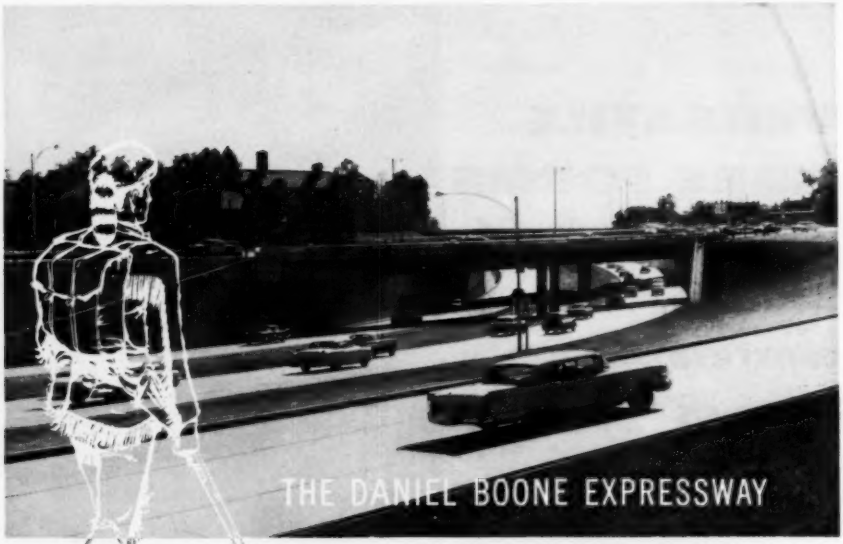

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This newly completed concrete highway bearing the name of Daniel Boone speeds traffic through the very countryside old Dan'l once traveled and knew only as a tangled wilderness.

The section of the Daniel Boone Expressway shown above was built by Millstone Construction, Inc., St. Louis.

Modern construction genius employing Laclede's complete designed reinforcement service is helping create a multi-million dollar national system of expressways, of which the Daniel Boone is a part. To withstand the constant pounding of cars to and from ever-expanding suburbs, these new freeways must be built *strong*.

Much of the needed strength comes from Laclede designed reinforcing steels—multi-rib reinforcing bars, welded wire fabric, welded dowel spacers, center and recess joints, tie bars and accessories.

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SAINT LOUIS, MISSOURI ◆ Producers of Steel for Industry and Construction

Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research
F. S. Fulton
Portland Cement Institute
Johannesburg, South Africa

Committee 209, Volume Changes and Plastic Flow in Concrete
Frederic Roll
University of Pennsylvania
Philadelphia, Pa.

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete
William H. Kuenning
Portland Cement Association
Skokie, Ill.

Committee 318, Standard Building Code
W. G. Plewes
National Research Council
Ottawa, Ont., Canada

Committee 334, Concrete Shell Structures
J. V. Christiansen
Worthington, Skilling, Helle and Jackson
Seattle, Wash.

Committee 336, Combined Footings
Andrew Reti
Civil Engineer
New York, N.Y.

Committee 401, Specifications for Structural Concrete
H. B. Britton
Department of Public Works
Albany, N.Y.

Committee 622, Formwork for Concrete
T. W. Hunt
Portland Cement Association
Chicago, Ill.

Victor F. Leabu
Giffels and Rossetti
Detroit, Mich.

Committee 716, High Pressure Steam Curing
S. E. Williams
Johns-Manville Products Corp.
Manville, N.J.

John C. Wycoff
Southern Lightweight Aggregate Corp.
Richmond, Va.

Akas named ACI advertising manager

Thomas C. Akas assumed the post of advertising manager of ACI on December 14. He succeeds Charles L. Cousins who resigned recently to join the advertising staff of the Pontiac Division, General Motors Corp.

A journalism graduate of Wayne State University, Mr. Akas served as assistant to the executive director of the Mail Advertising Service Association International, Detroit, during the past 3½ years. Mr. Akas also worked as a United Press staff correspondent in Detroit and as a copywriter for Farrand Publications. He is a member of Sigma Delta Chi, professional journalistic fraternity.

Oklahoma Chapter holds first annual meeting

The ACI Oklahoma Chapter held its first annual meeting on December 12 at the Biltmore Hotel in Oklahoma City with 46 members in attendance.

The afternoon session consisted of a business meeting and a technical session. Officers for the coming year were elected, committee appointments were made, and certain ammendments to the bylaws were discussed and voted to letter ballot. The technical session following the business meeting consisted of a panel discussion on reinforced concrete specifications and a talk by a

well known Oklahoma City attorney on the legal aspects of specifications.

ACI President Phil M. Ferguson was guest speaker at an evening dinner. Professor Ferguson, University of Texas, Austin, spoke about chapter organizations and affairs in general, and outlined some of the activities of the national organization. Of particular interest was his discussion of the ACI committee operation and how the local chapter might best serve in this area.

The 1960 officers of the Oklahoma Chapter are: president, Richard Jackson, Black and West, Architects, Tulsa, succeeding retiring chapter president Dan Hapke, Portland Cement Association; vice-president, Robert P. Witt, Oklahoma State University, Stillwater; secretary - treasurer, Stanley Gerlach, Gerlach Distribution Co., Oklahoma City; first director, Harold Wenzel, The Dolese Co., Oklahoma City; and second director, Ray Kraft, Refinery Engineering Co., Tulsa. Mr. Jackson and Professor Witt formerly served as vice-president and secretary-treasurer, respectively. The newly elected secretary-treasurer was formerly a director. Mr. Wenzel starts his second term as a director.

Highlights of chapter activities during the past year were two meetings held by the chapter to demonstrate the techniques of placing concrete slabs. One meeting was held in Oklahoma City where 158 people attended and the other in Tulsa with 163 in attendance.

Peruvian Cement launches educational program

Angel de Paz y Arroyo, general manager of Compania Peruana de Cemento Portland, Lima, Peru, has organized a program of lectures and publications aimed at better employment of cement and concrete in that area. The program is in direct charge of Jaime de las Casas, an ACI quarter-century-member-getter.

ACI Michigan members meet

About 60 ACI members and guests from the Michigan area met for a luncheon meeting on December 9, with William Krell, Portland Cement Association, acting as chairman.

Raymond C. Reese, consulting engineer, Toledo, Ohio, chairman of ACI Committee 318, Standard Building Code, was guest speaker at the meeting. Mr. Reese spoke on the topic of misapplications of the Building Code and discussed the work of Committee 318. After describing some first-hand encounters with misunderstandings in design or construction, he went on to explain that the Building Code is primarily a guide for the design of reinforced concrete by structural engineers. In addition to the written word, some knowledge and experience is required to properly and completely design the members of the structure required to adequately and economically accomplish the purpose.

ACI member James A. Nicholson. president of Toledo Plaster and Supply Co., Toledo, is scheduled to address the group at the February meeting. Mr. Nicholson is a past president of the Ohio Ready-Mixed Concrete Association and a director of the National Ready-Mixed Concrete Association. He will speak on the subject of ready-mixed concrete.

Although the Michigan group have not taken formal steps to organize as a chapter, regular monthly meetings are being held with about 50-75 ACI members and guests in attendance.

Kinzel appointed to National Research Council

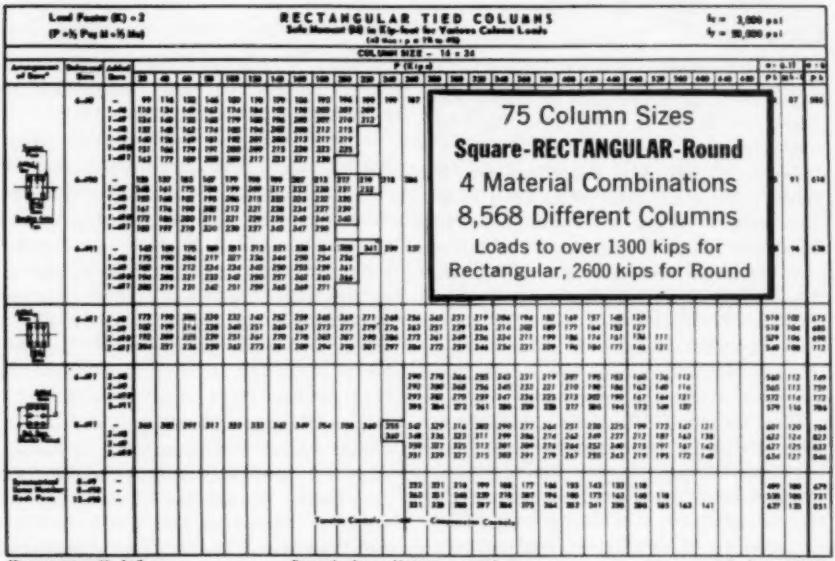
The National Academy of Sciences—National Research Council, Washington, D.C., has anounced the appointment of Augustus B. Kinzel as chairman of its Division of Engineering and Industrial

FELIX CANDELA says:

"I have examined carefully your preliminary draft on the Ultimate Design Column Tables and have found it extremely helpful and interesting . . . I believe both professions of architect and engineer will be most grateful to you when you publish these tables."

EDUARDO TORROJA says:

"I hope your book of Column Tables will enjoy the merit it deserves."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Research. Dr. Kinzel is vice-president for research of Union Carbide Corp.

Dr. Kinzel has received wide recognition for his research activities and has previously served the Academy—Research Council on a number of its committees and boards.

Magnel Medal goes to Esquillan

Prof. P. Malschaert. president of the Association of Engineers of Gand, Belgium, on Nov. 7, 1959, presented the first Gustave Magnel Gold Medal, for 1955-58, to Nicolas Esquillan for his project, the triangular plan arch of the Exposition Palace in Paris. The late Professor Magnel taught at the University of Gand and was director of the Reinforced Concrete Research Laboratory there which now bears his name. The awardee showed a color film of the construction of the exposition building after the ceremony.

The Magnel Gold Medal Award, established by the organization of engineering alumni of the University of Gand, will be conferred periodically to the designer of a building project incorporating an important and remarkable application of reinforced or prestressed concrete.

Engineers needed by federal government

The United States government needs engineers in various agencies and locations.

Examination announcement 211B issued Oct. 13, 1959, with no closing date, offers assignment in a number of specialized fields. The civil engineering option includes professional engineering work in all recognized fields of civil engineering such as structural, highway, bridges, and construction. Salaries range from $4490 for Grade GS-5 to $12,770 for Grade GS-15.

Application forms are available from any post office, U. S. Civil Service Commission office, or Boards of U. S. Civil Service Examiners for the Department of the Army or Bureau of Public Roads.

Expanded Shale, Clay and Slate Institute annual meeting

The Expanded Shale, Clay and Slate Institute held its 7th annual meeting the week of October 13 at the Royal York Hotel, Toronto.

The comprehensive shrinkage and creep program now under way at the National Bureau of Standards, Washington, D.C., was announced and discussed. This study will be conducted over a period of 5 to 10 years on concretes made from more than 20 aggregates produced by members of the Institute. Several additional research programs in the field of structural concrete, as well as concrete masonry, were discussed, and programs for future studies were approved.

The installation of officers, all of whom were reelected for a second year, followed the annual banquet. Officers are: president, Allan P. Taylor, Kenlite, Louisville, Ky., first vice-president, Cedric Willson, Texas Industries, Inc., Dallas; second vice-president, L. A. Thorssen, Consolidated Concrete Industries, Calgary, Alta., Canada; secretary, Lester Kennedy, Light Aggregates, Inc., Rapid City, S.D.; and treasurer, Ben F. Park, Buildex, Inc., Ottawa, Kan.

One of the features of the meeting was a panel discussion of structural concrete and concrete masonry, attended by over 110 architects and engineers, block manufacturers, and officials of Toronto and the surrounding area. The panelists were: Hector King, moderator, Cooksville-Laprairie Brick Ltd., Toronto; Jim Chandler, Chandler Materials Co., Tulsa, Okla.; Harold Hutchens, The Carter-Waters Corp., Kansas City, Mo.; Mark Huggins, University of Toronto; and Otto Frei, Georgia Lightweight Aggregate Co., Atlanta.

AN APPRECIATION

Charles Smith Whitney

1892 - 1959

When Charles Whitney, an internationally recognized consulting engineer of Milwaukee, died in Paris on Sunday, Oct. 25, 1959, the world lost one of its great men. The American Concrete Institute and the engineering profession lost one of its brilliant and devoted leaders.

Those who worked closely with Mr. Whitney, to all of whom he was known as "Charlie," remember him for his modesty and quiet manner. On these occasions it was hard to comprehend the far-flung scope of his engineering knowledge and professional services. In spite of the heavy demands made upon his talent, energy, and time throughout the world, he always served effectively as a member or chairman of many committees or as an officer of the Institute, finally becoming its President in 1955. Frequently to attend these meetings he had to return from a foreign country or to go immediately for a conference on important projects on which he served as a consultant.

In 1946 he became a member of the consulting firm of Ammann and Whitney, when they met as consulting engineers on plans for hangars at the Chicago airport and at New York International Airport in Idlewild. Among the structures on which he was design supervisor were Onondaga Memorial Auditorium in Syracuse, N.Y., the Alabama State Coliseum in Montgomery, and various hangars for American Airlines, Trans-World Airlines, and Pan-American World Airways. He was a consultant on many large Wisconsin projects, including the planning for the Milwaukee County Expressway system, the Lincoln Memorial Bridge, the structural designing of the Milwaukee Memorial Center, and a University of Wisconsin athletic building at Madison.

The firm of Ammann and Whitney, employing about 700 persons, had far-flung engineering offices and responsibilities which took Mr. Whitney to France, Greece, Iran, Ethiopia, Pakistan, Libya, and Turkey. The principal office is in New York City, but Mr. Whitney continuously maintained an office in Milwaukee.

At the time of his death, Mr. Whitney with Mr. Ammann, was engaged in the most "gigantic" of all his engineering projects, the "Narrows" bridges to be among the world's greatest, linking Brooklyn with Staten Island, both in New York City. The cost of the "Narrows" bridge is estimated to be $320 million.

A professional engineer in the strictest sense of the term, Charlie Whitney contributed generously of his talent and time in the preparation of publications for the American Concrete Institute and other national engineering societies. He was an active and contributing member of the American Con-

crete Institute, American Society of Civil Engineers, American Institute of Consulting Engineers, American Society for Testing Materials, Society of Professional Engineers, Wisconsin Society of Professional Engineers, International Association for Bridge and Structural Engineering, Institute of Aeronautical Sciences, the American Road Builders Association, and an honorary member of the Wisconsin chapter of the American Institute of Architects.

The influence of Charlie Whitney as an engineer and as an engineering scientist will be perpetuated by his many contributions to technical literature and committee reports. He was the author of *Bridges, A Study in Their Art, Science, and Evolution* and *Concrete Designer's Manual* (coauthor Hool). In 1925 the American Society of Civil Engineers awarded Mr. Whitney the James R. Croes Medal for his paper, "Design of Symmetrical Concrete Arches." His alma mater, Cornell University, conferred upon him, in 1925 and in 1927, the Fuertes Graduate Medal. For his many noteworthy contributions to its JOURNAL, the American Concrete Institute conferred upon him the Alfred Lindau Award in 1951, and the Wason Medal in 1932, 1952, and 1955. The Concrete Reinforcing Steel Institute conferred its first annual award to Mr. Whitney in 1949.

To measure the scope and breadth of Mr. Whitney's writings, the following ACI papers deserve mention:

"Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression" (*Proceedings* V. 33)

"Cost of Long-Span Concrete Shell Roofs" (*Proceedings* V. 46)

"Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures" (with Anderson and Salvadori, *Proceedings* V. 48)

"Reinforced Concrete Thin-Shell Structures" (*Proceedings* V. 49)

"Design of Blast Resistant Construction for Atomic Explosions" (with Anderson and Cohen, *Proceedings* V. 51)

"Year in Review" (*Proceedings* V. 52)

"Guide for Ultimate Strength Design of Reinforced Concrete" (with Cohen, *Proceedings* V. 53)

"Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members Without Shear Reinforcement" (*Proceedings* V. 54)

"Cantilevered Folded Plate Roofs ACI Headquarters" *Proceedings* V. 55)

As Past President of the Institute he served previously as President, Vice-President, and Director. He also served on Committee 318, Standard Building Code; the Standards Committee; and served as chairman of Committee 312, Plain and Reinforced Concrete Arches, 1930-1957; and as chairman of ACI-ASCE Committee 326. Shear and Diagonal Tension, 1950 to the present. Mr. Whitney was a member of the ACI Building Committee which successfully completed its assignment to provide the Institute's headquarters building.

Charles Smith Whitney was born in Bradford, Pa., Nov. 4, 1892. He was graduated from Cornell University, receiving the BS in CE degree in 1914

and the MS degree in 1915. In 1922 he opened his own office as a consulting engineer in Milwaukee. Mr. Whitney was an active member of St. Paul's Episcopal Church, serving as a vestryman and warden. He served in World War I with the Corps of Engineers on the construction of light railways in France.

Charles Whitney married Gertrude Schuyler, June 19, 1920. They had three children, Lillian Randolph of Jefferson, Wis., James Schuyler, deceased, and Charles Allen, a physicist at the Smithsonian Astrophysical Observatory in Cambridge, Mass.

By FRANK KEREKES in behalf of the ACI BOARD OF DIRECTION

Members named to ACI-CEB Collaboration Committee

ACI President Phil M. Ferguson has appointed Eivind Hognestad, Portland Cement Association, Skokie, Ill.; Raymond C. Reese, consulting engineer, Toledo, Ohio; and C. P. Seiss, University of Illinois, Urbana, to the American contingent of the ACI-CEB Collaboration Committee. Dr. Hognestad will serve as chairman of this group. Last April this committee attended the Comite Europeen du Beton meeting in Vienna on the invitation of CEB president Franco Levi, professor, Politecnico, Torino, Italy.

Professor Franco has also announced the names of those appointed to the ACI-CEB committee. These members will meet with their American counterparts or correspond with them concerning matters of mutual interest to the two organizations.

The permanent members designated for terms of 2 years are Professor Levi and George Wastlund, director, Cement and Concrete Institute, Stockholm, Sweden.

The supplemental members who will serve on the committee depending on the subject and the availability of the member are Prof. A. L. L. Baker, Imperial College, London, England; Prof. A. M. Haas, Technische Hogeschool, Delft, Netherlands; Dr. A. W. Hill, Cement and Concrete Association, London, England; Prof. Hubert Rusch, Technische Hogeschool, Munich, Germany; Dr. S. Soretz, Toristeg Steel Corp., Vienna, Austria; and Prof. E. Torroja, Instituto Tecnico de la Construccion y del Cemento, Costillares. Madrid (Chamartin), Spain.

Van Zelst elected to Cenco executive post

The election of Theodore Van Zelst as a vice-president of Cenco Instruments Corp. was announced recently. Mr. Van Zelst also was elected a director of Central Scientific Co., Cenco's principal manufacturing subsidiary. Mr. Van Zelst is president of Soiltest, Inc., which was acquired by Cenco in June, 1959.

Annual California Street and Highway Conference held at UCLA

Over 600 persons attended the 12th California Street and Highway Conference on the UCLA campus, Los Angeles, January 28-30. Victor W. Sauer, director of public works, Contra Costa County, served as general chairman. Ellis L. Armstrong, commissioner, U. S. Bureau of Public Roads, delivered the keynote address at the opening session.

This conference is presented annually by the Institute of Transportation and Traffic Engineering of the University of California with the location alternated between the Berkeley and UCLA campuses.

LOOKING AHEAD

Feb. 1-5, 1960 — Committee Week, American Society for Testing Materials, Hotel Sherman, Chicago, Ill.

Feb. 15-19, 1960 — Annual Convention and Exposition, National Sand and Gravel Association, Conrad Hilton Hotel and Coliseum, Chicago, Ill.

Feb. 22-24, 1960 — Annual Convention and Exposition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago, Ill.

Mar. 7-10, 1960 — 52nd Annual Convention, American Concrete Pipe Association, Fairmont Hotel, San Francisco, Calif.

Mar. 7-11, 1960 — New Orleans Convention, American Society of Civil Engineers, Jung Hotel, New Orleans, La.

Mar. 14-17, 1960 — 56th Annual Meeting, American Concrete Institute, Commodore Hotel, New York, N. Y.

May 4-8, 1960 — Cleveland Convention, American Society of Civil Engineers, Hotel Cleveland, Cleveland, Ohio.

June 26-July 1, 1960 — Annual Meeting and Apparatus Exhibit, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

July 11-18, 1960 — Second World Conference on Earthquake Engineering, Science Council of Japan, Tokyo and Kyoto, Japan.

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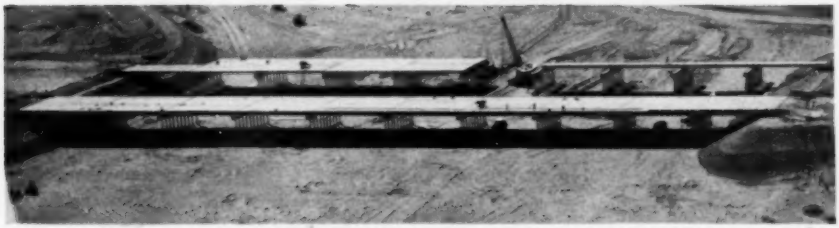
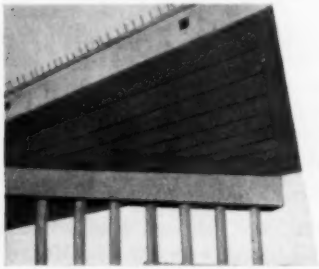
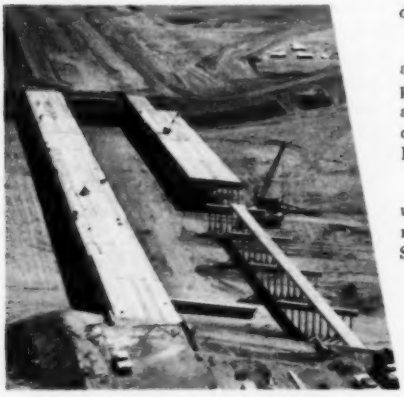


Dual lane highway bridges, constructed with prestressed pretensioned Amdek Beams, near completion in Socorro County, New Mexico. Erected for the New Mexico State Highway Department as part of the Federal Interstate Program.



Low depth-to-span ratio means less weight, greater clearance and reduced fill. Smooth underdeck offers minimum resistance to flood debris.



200 Amdek Box Girders used to build 506 foot bridges on 30° skew. Independent dynamic and static load tests prove superior strength of Amdek Skew Beams.

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Fink retires

Joseph M. Fink recently retired as chief of the structural department of the Department of Building and Safety Engineering of the City of Detroit. Mr. Fink was with the department since 1946 and was chief structural engineer since 1952.

In 1958 he received the first annual Huron Achievement Award from the Concrete Improvement Board of the Metropolitan Area of Detroit for his outstanding contributions to the improvement of concrete in the metropolitan Detroit area. He was cited particularly for his efforts toward adoption of the new Detroit building code.

Since graduating from Yale University in 1916, excluding 2 years in the army during World War I, his work has been equally divided between private industry and municipal employment.

Mr. Fink is a member of ACI and a number of other technical societies. He expects to engage in private consulting work.

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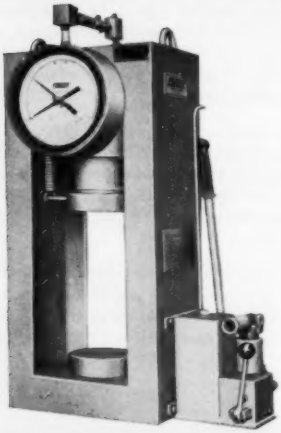

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Furane opens New York plant

Furane Plastics Inc., Los Angeles, producer of epoxy resins, has opened a plant in Hempstead, Long Island, N.Y. Leon Wynn will serve as eastern sales manager and will be in charge of the operation.

PCA elects five new directors

The Portland Cement Association at its annual meeting in Chicago, November 18, elected five members to its board of directors.

The new directors are: Chester S. Crawford, president, Whitehall Cement Manufacturing Co., Philadelphia; L. R. Forbrich, general manager, Green Bag Cement Division, Pittsburgh Coke and Chemical Co., Pittsburgh; Ellroy King, president, Halliburton Portland Cement Co., Corpus Christi; Frank B. Warren, president, Bessemer Limestone and Cement Co., Youngstown, Ohio; and W. S. Ziegler, president, Saskatchewan Cement Co., Ltd., Regina, Sask., Canada.

Retiring directors are: Albert M. Andreas, chairman of the board. North American Cement Corp., New York; B. F. Fox, vice-president and general manager, British Columbia Cement Co., Ltd., Victoria, B.C., Canada; C. T. Fuller, chairman of the board, Allentown Portland Cement Co., Allentown, Pa.; L. T. Welshans, general manager, Standard Portland Cement Division, Diamond Alkali Co., Cleveland.

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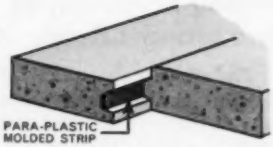
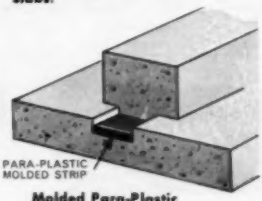
Because it presents sealing surfaces on all sides, Molded Para-Plastic is especially effective in sealing keyed construction joints and foundation footings. Not recommended for use in joints where the surface of the material is exposed, or when hydrostatic pressure is present. Available in various sizes with thickness ranging from ¼" to 1" and widths from 1 to 12 inches. Standard length is 5 feet.



Keyed construction joint in vertical walls and horizontal slabs.



Molded Para-Plastic sealing at a footing.

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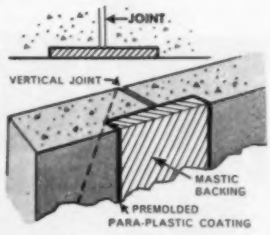

Application of Para-Lateral Sealing Strip to rear face of wall at joint.

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University of Mississippi holds highway conference

The University of Mississippi conducted the sixth annual Mississippi highway conference on February 11-12 on the campus at the Mississippi Center for Continuation Study.

Panel discussions were devoted to highway design, recent developments in highway construction, and new developments in highway engineering. Discussion groups covered various phases of highway design.

Master Builders open two branch offices

The Master Builders Co., Cleveland, has announced the opening of offices in Charlotte, N.C., and Salt Lake City, Utah.

Edwin A. Decker will serve as manager of the Charlotte branch office. He joined Master Builders in 1954 and has served as a sales representative and field engineer until his present appointment. Martin J. Switters will serve as field representative for the Salt Lake City branch.

Soiltest appoints Davies manager in Europe

John Robinson Davies will head Soiltest's operations in Europe, the Middle East and Africa, with headquarters in Laususanna, Switzerland. A graduate of Oxford University, England, Mr. Davies has had many years of experience in the scientific apparatus industry.

Soiltest has established a sales office in Brussels, Belgium, and a distribution center and stocking warehouse in Breda, The Netherlands.

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Honor Roll

January 1 — January 31, 1960

The number of members, to date, who have proposed one or more new members since last year represents 0.005 percent of our total membership.

If each of ACI's 10,103 members would introduce but one new member to the Institute, we would soon double our membership. Put your name on the Honor Roll next month.

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Alfonso Marin E.	27
Joaquin Spinel L.	6
James A. McCarthy	4
J. E. Backstrom	3
Ira M. Beattie	3
George B. Begg, Jr.	3
Alan Carter	3
Fernando de Angulo	3
Benjamin P. Felix	3
Phil M. Ferguson	3
Frank D. Gaus	3
Hans Gesund	3
Fabian Guerra	3
Samuel Hobbs	3
Robert J. Kadala	3
T. C. Kavanagh	3
Clyde E. Kesler	3
E. V. Konkel	3
William J. Krefeld	3
John C. McCoe	3
Luis F. Magrina	3
W. E. Moulton	3
Russell Porter	3
Frank A. Randall, Jr.	3
Salvador Rodriguez	3
Howard J. Rosenberg	3
H. Rusch	3
Herbert A. Sawyer, Jr.	3
H. M. Schwartz	3
A. L. Small	3
George B. Southworth	3
D. A. Van Horn	3
Ellis S. Vieser	3
Cedric Willson	3
Robert P. Witt	3
Mark R. Woodward	3
Jay antilal S. Alagia	2
Arthur H. Andersen	2
Pedro M. Bassim	2
W. S. Cottingham	2
Ward W. Engle	2
Gregorio Hernandez	2
Fred W. Jacobs	2
Simon Lamar	2
R. R. Neal	2
Raoul E. Pallais	2
F. K. Taskin	2
Rogelio Bonilla Torres	2
Leon A. Yacoubian	2

New Members

The Board of Direction approved applications in the following categories: 91 Individual, 3 Corporation, 19 Junior, and 27 Student, making a total of 140 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on Dec. 1, 1959, was 10,103.

INDIVIDUAL

ABNEY, GEORGE H., Dallas, Tex. (Struct. Engr.)

ADKINSON, D. J., Hamilton, Ont., Canada (Sales Mgr.)

BAKLEY, C. NEWTON, Erie, Pa. (Vice-Pres. & Gen. Mgr., Ready Mixed Concrete Co.)

BAUER, EDWIN L., Honolulu, Hawaii (Arch.)

BEENHAM, TERENCE WILLIAM, Romford, England (Struct. Engr., Simon-Carves, Ltd.)

BOFF, PAUL, New Kensington, Pa. (Tech. Dir., Burrell Constr. & Supply Co.)

BOICE, HOWARD E., Phoenix, Ariz. (Chf. of Opers., Del E. Webb Constr. Co.)

BRACAMONTES, LUIS, Mexico D. F., Mex., (Under Sec. of Pub. Wks., Mexican Gov't)

BRUTON, JOHN M., Indianapolis, Ind. (Struct. Engr., Carl M. Geupel Constr. Co.)

CAHN, W. J., Chicago, Ill. (Chf. Engr., Herlihy Mid-Continent Co.)

CHILTON, DONALD W., Huntington Sta., N. Y. (Quality Conc. Engr., PCA)

CORRIGAN, E. W., Chicago, Ill. (Gen. Mgr., Great Western Contracting Co.)

COTE, JEAN-R., Rouyn, Que., Canada (Cons. Engr.)

CROWDER, C. DEWEY, JR., Mobile, Ala. (Assoc. Arch. & Engr., Platt Roberts & Co.)

CUMMINGS, T. F., Odessa, Tex. (Plant Chem., Southwestern Portland Cement Co.)

CUMMINS, ROBERT J., Houston, Tex. (Cons. Engr.)

EPPES, BILL G., Austin, Tex. (Asst. Prof., Engrg. Mech., Univ. of Tex.)

ESCALANTE PORTAS, CARLOS, Mexico, D. F., Mex. (Struct. Engr.)

FLACK, HARRY L., Denver, Colo. (Matls. Engr., USBR)

FRANCESCHINI, JAMES L., Toronto, Ont., (Vice-Pres., Dual Mixed Conc. & Matls., Ltd.)

FREIMAN, JACK, Bloomfield, Conn. (Gen. Mgr.)

GIRAULT, MANUEL, Mexico, D. F., Mex. (Constr. & Design Engr., Bufete Industrial)

GRANDLE, EDWARD D., Maracaibo, Venezuela (Design Engr., Constructora)

GRAY, GEORGE A., Blacksburg, Va. (Prof., Civil Engr., Va. Polytech)

HAMM, ROLLAND CHARLES, Milwaukee, Wis. (Dist. Engr., Amer. Inst. of Steel Constr., Inc.)

HANSEN, KENNETH D., Denver, Colo. (Proj. Engr.)

HECKER, HAROLD M., Rochester, N. Y. (Cons. Engr.)

HEFTER, HARRY O., Chicago, Ill. (Struct. Engr.)

HORN, A. D., Hartford, Conn. (Chf. Engr.)

HUBBARD, WARREN M., Riverside, Cal. (C. E., Riverside Cement Co.)

INCERA GARDEAZABAL, JOSE, Mexico, D. F., Mex. (Cons. Engr., Comision Federal de Electric)

INNAMORATO, SAVERIO, White Plains, N.Y. (Bldg. Insp., City of White Plains)

JENSEN, O., Paris, France (Asst. Engr., Bureau D'Etudes ECB)

JOINER, JAMES R., Dallas, Tex. (Partner, Hunt & Joiner)

KNOWLTON, HOOPER, JR., Salt Lake City, Utah (Partner, Knowlton & Jones, Cons. Engrs.)

KORDINA, KARL, Braunschweig, Germany (Prof. Braunschweig Techn. Hochschule)

LAUCOMER, FRANK, Birmingham, Mich. (Struct. Engr., Laucomer & Manser)

LEGGAT, A. M., Terapa, Hamilton, New Zealand (Engr., Mgr., Ready Mixed Conc. Supplies Ltd.)

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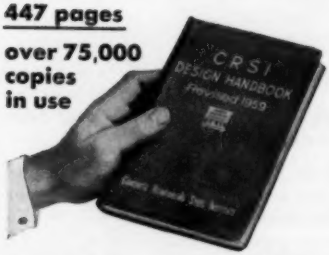

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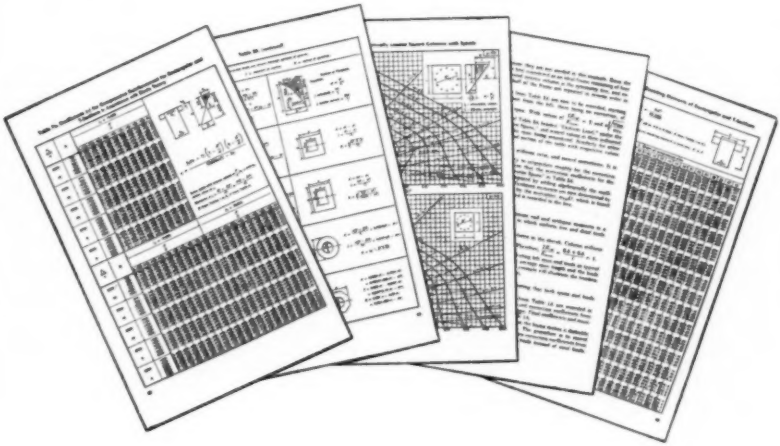

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